

Durability of Building Materials & Components 8

Service Life and Asset Management

VOLUME

1

**Service Life and
Durability of Materials
and Components**

**edited by
MICHAEL A. LACASSE
and
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Dr. Michael A. Lacasse is a materials scientist and building science engineer who since 1991 has worked as a Research Officer at the Institute for Research in Construction, of the National Research Council Canada. His work focuses on the development of methods to assess the long-term performance of building materials and components and the durability of building envelope assemblies. Since joining the IRC, he has been active in various CIB and RILEM technical committees related to performance, durability and service-life prediction. He currently co-ordinates the activities of the joint CIB/RILEM technical committee W080/175-SLM in which issues related to the prediction of service life of building materials and components are reviewed.



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**Volume One
Service Life and Durability
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Durability of Building Materials and Components 8

Service Life and Asset Management

Proceedings of the Eighth International Conference
on Durability of Building Materials and Components, 8dbmc

Vancouver, Canada
May 30 - June 3, 1999

Volume One
**Service Life and Durability
of Materials and Components**

Edited by

Michael A. Lacasse and Dana J. Vanier
*Institute for Research in Construction,
National Research Council of Canada*

NRC-CNRC

NRC Research Press
Ottawa, 1999

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Printed on acid-free paper

Publisher's Note: These books have been prepared from camera ready copy provided by the editors in order to make the books available for the conference.

ISBN 0-660-17737-4

NRC No. 42738

Canadian Cataloguing in Publication Data

International Conference on Durability of Building Materials and Components (8th : 1999 : Vancouver)

Durability of Building Materials and Components 8 : Proceedings of the Eighth International Conference on Durability of Building Materials and Components

Complete in 4 vols.

Includes an index.

"Vancouver from May 30 to June 3, 1999"—Introd.

Issued by the National Research Council of Canada.

ISBN 0-660-17737-4 (v.1)

ISBN 0-660-17741-2 (v.2)

ISBN 0-660-17742-0 (v.3)

ISBN 0-660-17743-9 (v.4)

1. Building materials — Service life — Congresses.

I. Lacasse, Michael A., 1953- . II. Vanier, D. J. (Dana J.) III. National Research Council Canada. IV. Title.

TA401.3156 1999

691.

C99-980130-9

Inquires: Proceedings Program, NRC Research Press, National Research Council of Canada, Ottawa, Ontario K1A 0R6, Canada

Correct citation: Lacasse, M.A. and Vanier, D.J. (Editors) 1999. *Durability of Building Materials and Components 8: Service Life and Asset Management, Volume One: Service Life and Durability of Materials and Components*. NRC Research Press, Ottawa, Canada, 898 pages.

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Preface

Asset managers are faced with many difficult decisions regarding when and how to maintain and repair their constructed facilities. The reasons behind these difficult decisions are the growing fiscal constraints, an increasing maintenance deficit and a substantial repair backlog. In addition, asset managers' resources are being challenged from all sides: they are also being asked to cut cost, privatise operations, outsource responsibilities, reduce maintenance and increase efficiency. These problems are exacerbated by the lack of: (1) usable data, (2) information and knowledge related to maintenance, (3) repair and service life prediction techniques, and (4) tools to assist the asset manager to make proper inspection, maintenance, repair and replacement choices.

Many of these problems would be solved if more information exchange were present in the construction industry regarding service life and asset management. Many organisations have in recent years recognised the scope of the problems facing asset managers. Unfortunately existing opportunities offer only partial solutions to many of the problems in asset management. Hence there is a pressing need for research and development in this field. For example: (1) what maintenance data are required to predict service life of materials and components, (2) what are the life cycle costs of maintenance and repair alternatives, (3) what techniques are available for service life prediction, (4) how can performance be quantified using objective and representative indicators (5) how can the likelihood of failure be calculated and the consequences of failure be identified, and finally, (6) how to integrate this information to allow asset managers to make cost-effective and objective decisions on maintenance, repair and renewal alternatives?

It is within this framework that the conference themes were chosen and to which many authors have provided significant contributions to the advancement of their respective research fields. The collected body of works, presented herein, focuses on topics related to integrating existing knowledge together with the more recent contributions on service life and durability of construction and building materials, components, and assemblies. Particular emphasis has been placed on information technologies in construction and its' role in integrating knowledge through classification schemes, product models, information retrieval systems and collating and dissemination technologies.

xxx *Preface*

The proceedings have been published in four volumes. The first volume, "Service Life and Durability of Materials and Components" contains four parts each reflecting those contributions made to the durability of specific building materials, namely: concrete, masonry, wood, and polymers. Within the part on concrete materials, there are several sub-sections including service life prediction of concrete structures, strength and durability, corrosion, and papers on issues related to design and innovation. The second volume "Durability of Building Assemblies and Methods of Service Life Prediction", contains six parts, the first three of which provide those papers related to environmental characterization, building facades, and roofing materials and components. Within this last area, roofing maintenance management has likewise been included as this provides the collected works in the area of roofing as a distinct part of the proceedings. The final three parts of this volume focus on methods of service life prediction, including probabilistic and reliability methods, generic methods, and lastly, development and use of guides and standards. "Performance, Service Life Prediction and Sustainable Construction" is the third volume in which the collected works are presented in four parts, starting with maintenance management, and followed by life cycle costing and economics, performance and serviceability of buildings (CIB W060 workshop), and life cycle analysis and sustainable, durable design. The final volume "Information Technology in Construction", represents those papers submitted to the CIB W78 workshop of the same name and is prepared in eight sections including computer-integrated construction, the construction process, decision-support systems, design systems, life cycle models, product modeling, process modeling, and virtual reality.

Collectively these contributions provide a selected overview of approaches, methods, techniques, tools, systems and technology used to develop knowledge on the service life durability of construction and building materials, components and assemblies and the integration of this information towards asset management. As such, it is hoped the many who have contributed to these proceedings, as well as those who in the future will partake in perusing its many pages, will gain in an understanding of the issues, a knowledge of the techniques, some insight into the research, and healthy discernment of useful technologies.

Finally, in editing the various papers and thereafter preparing them for final copy, the editors have made every effort to produce a faithful reflection of the authors' contributions based on their revised manuscripts. However, as might be expected in proceedings of this size, and irrespective of their many efforts, the editors concede that unfortunately, but ultimately, there

are likely to be found errors and omissions. These we genuinely regret and would like to express in advance our apologies for any such oversights.

M.A. Lacasse and D.J. Vanier,
Editors, 8DBMC
Ottawa, 1999

Foreword

This conference represents the 8th triennial in a series, in which the previous seven were held in Ottawa (1978), Gaithersburg (1981), Espoo (1984), Singapore (1987), Brighton (1990), Omiya (1993), and Stockholm (1996). The evolution of these collected works over the past two decades attest to the persistent and incessant pursuit of knowledge on the durability of materials, components and assemblies whether in regards to fundamental or applied research, or the most recent technological innovations. These conference proceedings have been representative of the state-of-the-art on service life prediction methods and the durability of constructed facilities.

More recently, attention has been placed on deriving from these sources of information, a means of providing practitioners with ready access to the vast body of knowledge encompassed within studies on the "durability of materials". Moreover, recent advances in information technologies permit its use as an integration tool and dissemination device, and a suitable interface to the current and evolving needs of practitioners. It is in this context that the 8DBMC provides a unique venue for exhibiting the most recent innovations in information technologies as a tool to link various research fields associated with service life prediction.

Consequently, this set of conference proceedings focuses on topics related to integrating existing knowledge together with more recent contributions on service life and durability of construction and building materials, components and assemblies. This includes work in the area of service life prediction methods and modeling, field studies and evaluation methods, degradation mechanisms, material characterization, and other related items. It is also concerned with providing a forum for new knowledge in the area of service life directly related to life cycle economics, maintenance management, performance requirement modeling, probabilistic and reliability methods, as well as, promoting the integration of this knowledge towards practical applications with emphasis on the use of information technologies as an integration tool.

These proceedings will be of interest to construction researchers, practitioners and designers concerned with service life prediction and durability as well as asset managers interested in obtaining knowledge about the most recent innovations currently being considered as management tools. Information technologists will find considerable scope towards the application side of their emerging technologies.

Foreword xxxiii

These volumes are therefore a singularly valuable set of documents and a worthy addition to the previous collected works and to the annals of the Institute for Research in Construction.

Sherif Barakat
Director General
Institute for Research in Construction
National Research Council Canada
Ottawa, 1999

Acknowledgements

The preparation of this set of four volumes represents a considerable amount of focused effort by a good many capable, dedicated, and resourceful people. In this regard the efforts of those who participated in its completion must be acknowledged. Hence we are indebted to Mr. Stacey Nunes for his singular efforts co-ordinating the review and processing of papers and, to Mr. Daniel Payer for his keen and assiduous work in preparing the manuscripts for publication format. As well, our sincere gratitude is extended to Mr. Pierre Lamoureux and Ms. Marie-Sophie Bourque of the 8DBMC Conference Secretariat who both insured the timely processing of the many contributions. We would also like to extend our appreciation to those at the NRC Research Press who have worked so diligently to prepare the collected proceedings for publication and imprint. In particular, mention is made of the combined efforts of Mr. Gerry Neville, Ms. Evelyn M. Kidd and Mr. Robert M. Forrest. Our appreciation is also extended to Mr. Marc Y. Boileau of the NRC Reprographic Services. Finally, this publication is made possible by the authors and also, the many and often un-proclaimed efforts of the reviewers. This collective of experts whose dedication and combined knowledge in reviewing the papers must likewise be acknowledged. The level of quality of this NRC publication is a direct result of their respected opinions.

M.A. Lacasse and D.J. Vanier
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Ottawa, 1999

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PART ONE

**DURABILITY OF CONCRETE
MATERIALS AND
STRUCTURES**

**SERVICE LIFE PREDICTION OF
CONCRETE STRUCTURES**



1 SERVICE LIFE DESIGN FOR THE WESTERN SCHELDT TUNNEL

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Abstract

Due to the high construction costs and the social importance the durability demands for large infrastructures are becoming more and more important. Service life requirements of 100 year or even more are usual. For the bored reinforced concrete tunnel under the Western Scheldt in the Netherlands the requirement was a service life of at least 100 years. No method had been specified to prove this service life. Since the concrete codes are only based on deem-to-satisfy rules for the durability, without any specification to the service life, it was not possible to base the design on existing codes. The service life design has been made on basis of the methodology that has been developed in a research project of the European Community. This project with the name 'DuraCrete' has further improved the existing reliability and performance based structural design method by introducing the modelling of degradations and environmental actions. It is believed that the service life design of the Western Scheldt Tunnel is the first project where the DuraCrete approach has been applied in practice.

Keywords: concrete, durability, tunnel, probabilistic design

1 Introduction

Tunnels in the Netherlands are traditionally built by prefabricating large tunnel sections. These sections are shipped to the final location and scuttled. Recently, another construction method is applied: boring tunnels. The main difficulties in boring in soft soils like clay and sand have been overcome. The second bored tunnel in the Netherlands is now under construction. The tunnel consists of two tubes with

an external diameter of 11 m and a length of 6.5 km. The tunnel will cross the Western Scheldt in the south-western part of the Netherlands.

The bored tunnel sketched in Figure 1 as a cross-section and in a longitudinal perspective has to be designed against variously induced re-bar corrosion. The bored tunnel is located in chloride contaminated soil and therefore under chloride attack. The inside walls of the tunnel are not ventilated and therefore under the of carbonation. As well, they are exposed to road traffic induced chloride contaminated salt fog and splash environment. Furthermore, leaking joints will eventually lead to chloride attack within the joints and partially at the inside surfaces in particular for deep points of the tunnel.

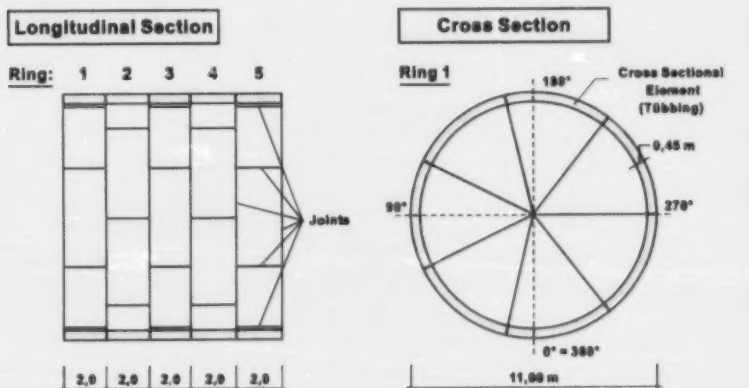


Fig. 1: Geometry of the bored tunnel under the Western Scheldt

The cross-section of a singular circular segment and the expected environmental loading is presented in the following Figure 2.

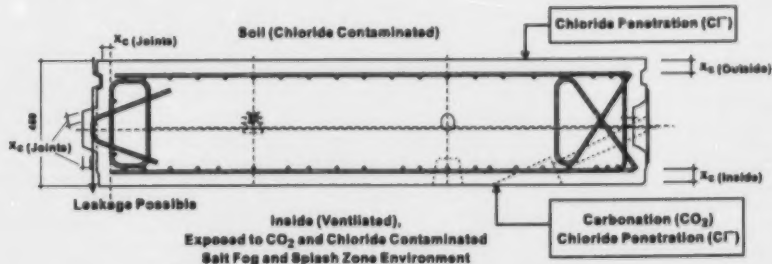


Fig. 2: Geometry of the reinforced circular segment of the Western Scheldt bored tunnel and expected environmental loading

In the contract between Rijkswaterstaat (investor) and the Combinatie Middelplaat Westerschelde (KMW, contractor) a service life of at least 100 year has been fixed, without a specification of the related performances and the reliability level. Schiessl · Raupach · Consulting · Engineering and TNO have been consulted on the service life approach.

2 Historical overview of service life design in practice

The present design approach with respect to durability of concrete structures is based on a reasonable understanding of the main degradation processes for concrete, reinforcement and prestressing steel. The design is however not explicitly formulated as a service life. It is based on deem-to-satisfy rules (for example minimum cover, maximum water/binder ratio and crack width limitation) and the assumption that if these rules are met, the structure will achieve an acceptably long but unspecified life. The information about the service life to be achieved is for a large extent empirical. Improving the durability results in increasing building costs without any quantification of the reduction of maintenance costs or failure costs. Current design methods permit only the calculation of whole life cycle costs from assumptions with respect to maintenance and failure rates. Thus, there are no objective means for demonstrating that future maintenance and repair costs will be acceptably low.

This common design approach to durability has other disadvantages. The rules are inadequate in some aggressive environments, while they are too rigorous in other environments. In some cases, this results in a 'belts and braces' approach (many different types of measures on top of each other) which may contain unnecessary and even counteractive measures.

A lack of durability can cause serious safety and serviceability problems for structures. Despite this, at this moment designers have considerably more attention for load and resistance based structural design than for durability design. Recent history has however shown that due to a lack of durability various types of damages and in single cases even collapses can occur with tremendous overall costs for repair.

For large infrastructures the requirements with respect to durability are in general more stringent than the codes. Some examples of storm surge barriers in the Netherlands can demonstrate this. For the Haringvlietsluizen (1965) the requirements were based on the experts opinion of that time. The concrete cover was raised to 70 mm, the water/cement ratio was restricted to 0.45 and blast furnace slag cement was used as binder. For the Eastern Scheldt Barrier (1985) the durability requirements were for the first time expressed in terms of time. A service life of 200 years has been required. For the concrete cover this was not feasible. A mean service life of about 85 years has therefore been accepted. After that period the concrete cover will be replaced. For the Maeslantkering (1995) a service life of 100 years has been specified. The concrete cover of this barrier has been enlarged with a factor $\sqrt{100/50} = \sqrt{2}$. The figure 100 is the required service life and the figure 50 is the service life that is assumed in the Dutch concrete code. In none of these designs has a proper approach for predicting the service life been applied.

3 Design framework

One of the most important reasons that no proper service life designs were made was the lack of a methodology and a design framework. In the past period this has been developed and will be developed further. Some major steps are the CEB Bulletin 238 (Schiesl *et al.* 1997) and the Brite/Euram project 'DuraCrete' that has been finished in February 1999 (TG7 1999). The essential items of the methodology have been copied from the structural design (Siemes and Rostram 1996):

- the performances are related to limit states
- the reference period is similar to the design service life
- the reliability index is used to reduce the failure probability.

The new items in the approach are the introduction of environmental actions and models describing the degradation of the concrete and the reinforcement.

The limit states that have been defined in the structural codes are the serviceability limit state (SLS) and the ultimate limit state (ULS). If a SLS is exceeded the functioning of the structure is restricted; e.g. the deflection of a beam is more than 20 mm. If an ULS is exceeded the structure has collapsed, fractured, turned over, etc., the static equilibrium has been lost and safety is endangered.

The limit states of the structural codes may be extended with other limit states. For example the watertightness of a tunnel can be expressed as a SLS (for example: the leakage water hinders the traffic in the tunnel) or as an ULS (for example: the leakage water floods the tunnel). Also new types of limit states may be introduced. For example the limit state to reduce the amount of repair. In case of corrosion of the reinforcement it is possible to reduce the amount of repair by requiring that no onset of corrosion may occur.

In Figure 3 an example of a service life distribution is given. The main parameters in such a distribution are the mean value μ (magnitude parameter) and the standard deviation σ (scatter parameter). Instead of the standard deviation we can also use the variation coefficient $V = \sigma/\mu$. By means of the reliability index, β , the probability that the service life is lower than the design service life L_d is identified.

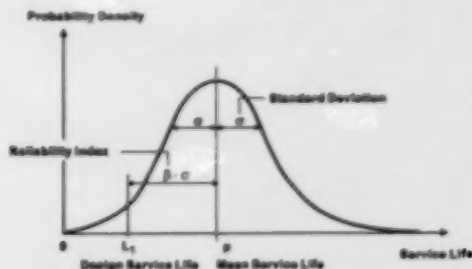


Fig. 3: Example of a probability density function for the service life

With every performance an unique distribution function is connected, depending on the geometry of the structure, material properties, mechanical loads and the environmental actions. This implies that for every relevant performance the distribution function has to be established, the design service life as well as the reliability index must be defined. In this respect it is important to define a limit for the performance that separates the adverse state from the required state.

In Table 1 some values for the reliability index, β , and corresponding approximate failure probabilities are given (European Code EC1 and Dutch Code NEN 6700). These reliability indexes can be used if lack of durability leads to an event that leads to an unacceptable loss of serviceability or to a loss of structural safety (Siemes and Rostam 1996). If failure leads only to economical consequences serviceability limit states (SLS) are applied, e.g. onset of corrosion. If failure leads to severe consequences, e.g. (total loss of the structure or victims) ultimate limit states (ULS) are applied (collapse due to overloading or due to excessive material degradation). Other intermediate stages may be defined by the owner (investor) in order to reduce repair costs, ensuring the structural appearance, etc..

Table 1: Examples of some reliability indexes in structural codes

Type of Performance	Reliability Index β in a Period of 50 Year		Approximate Failure Probability in 50 Years
	EuroCode (EC1, 10/1994)	Dutch Code (NEN 6700, 04/1991)	
ULS	3.8	3.6	10^{-4}
SLS	1.5	1.8	10^{-2}

4 Design calculations

4.1 Introduction

In this paper we will restrict to the design of the tunnel lining, although in practice the whole tunnel has been based on a service life design. The following limit states were chosen:

- Onset of corrosion
- Corrosion induced spalling and corresponding loss of watertightness
- Collapse of the structure.

Onset of corrosion is a serviceability limit state. When corrosion has started, maintenance is much more complicated to perform and therefore much more expensive than for a structure without initial corrosion activity. Consequently there is an economic risk present. Corrosion induced spalling in the area of joints can lead to a failure in watertightness. In dependency to the failure probability the operating costs will increase (pumps, etc.) to an upper limit. The highest economic damage at the serviceability level is reached when the leakage rates are so high that the tunnel can not be operated. The collapse of the structure should be avoided at a highest

reliability level (ultimate limit state, ULS) not only to avoid with corresponding low failure probabilities the highest economic damage, but also to save human life.

Finally, the following set of operational limit states and their related reliability indexes was derived (Table 2).

Table 2: Set of operational limit states

Event	Limit State	Reliability Index •
Onset of corrosion	SLS and repairable	1.5 - 1.8 EC1 - NEN 6700
Corrosion induced spalling and corresponding failure in watertightness	SLS and repairable	2.0 - 3.0 Proposal
Collapse of the structure	ULS	3.6 - 3.8 NEN 6700 - EC 1

4.2 Design example: Concrete cover within the joints

Due to structural design constraints the concrete cover within the joints should be minimised. Therefore, the joint with its restricted cover of $x_{c(joint)}$ was identified as the most critical detail with regard to durability. In the case of permanent leakage, the concrete is subjected to chlorides coming from the chloride contaminated soil. Constantly humid conditions are assumed; carbonation can be neglected. The identified deterioration mechanism is in this case the chloride induced corrosion. The question that has to be answered here as an example is: Which material resistance against chloride penetration in combination with which concrete cover x_c is necessary in order to fulfil the investors requirements? In the following paragraph the carried out design of the concrete cover $x_{c(joint)}$ is presented for a serviceability limit state (Onset of corrosion: $T_{Service\ Life} = 100$ year with a minimum reliability index of $\beta_{0,SLS} \geq 1.50 - 1.80$).

4.3 Deterioration model

In the case of a chloride induced reinforcement corrosion, deterioration models are required, describing the initiation period as well as the propagation period. As mentioned before, here we will only discuss the initiation period.

The following deterioration model is describing the time dependent process of chloride penetration until the depassivation of the re-bar will occur. The suggested model to predict the duration of the initiation period in the case of chloride induced re-bar corrosion has been identified within an intensive literature research carried out in a Brite/EuRam-project DuraCrete (Alisa et al. 1998), Equations (1), (2) and (3)):

$$x(t) = 2 \cdot C_{(crit)} \cdot \sqrt{k_i \cdot D_{RCM,0} \cdot k_e \cdot k_c \cdot \left(\frac{t_0}{t}\right)^n} \cdot t \quad (1)$$

with:

$$k_i \cdot D_{RCM,0} = D_0 \quad (2)$$

$$C_{(CH)} = \text{erf}^{-1} \left(1 - \frac{C_{CH}}{C_{SN}} \right) \quad (3)$$

with:

- x_c : concrete cover in [mm]
- D_0 : effective chloride diffusion coefficient at defined compaction, curing and environmental conditions, measured at time t_0 in [m^2/s]
- $D_{RCM,0}$: chloride migration coefficient at defined compaction, curing and environmental conditions, measured at time t_0 in [m^2/s]
- C_{CH} : chloride threshold level in [wt.-% Cl/binder]
- n : factor which takes the influence of age on measured material property into account
- k_i : constant parameter which transfers the measured chloride migration coefficient $D_{RCM,0}$ into a chloride diffusion coefficient D_0 , k_i
- k_e : constant parameter which considers the influence of environment on D_0
- k_c : constant parameter which considers the influence of curing on D_0
- erf^{-1} : inverse of the error function
- C_{SN} : surface chloride level in [wt.-% Cl/binder]
- t : exposure period in [year],
- t_0 : reference period in [year], in this case $t_0 = 28$ day

4.4 Data

The data for the service life design performed for the tunnel segments are given in the following Table 3 (the required variables under consideration can be drawn out of Equation (1) and (3)):

Table 3: List of stochastic asvariables influencing the duration of the initiation period

Variable No	Parameter	Dimension	μ	σ	Distr. Type
1	x_c - Concrete Cover	[mm]	37	2	Expon. Distr.
2	$D_{RCM,0}$ - Cl-Migration Coef.	[$10^{-12} \text{m}^2/\text{s}$]	4.75	0.71	Normal Distr.
3	C_{CH} - Critical Chloride Content	[wt.-%/binder]	0.70	0.10	Normal Distr.
4	n - Age Exponent	[-]	0.60	0.07	Normal Distr.
5	k_i - Factor Test	[-]	0.85	0.20	Normal Distr.
6	k_e - Factor Environment	[-]	1.00	0.10	Normal Distr.
7	k_c - Factor Execution	[-]	1.00	0.10	Normal Distr.
8	C_{SN} - c(Cl) - Concrete Surface	[wt.-%/binder]	4.00	0.50	Normal Distr.
9	t_0 - Reference Time	[year]	0.0767	-	Deterministic

In the following it will be shown for the first three listed stochastic variables how each distribution function was obtained.

4.4.1 Concrete cover

During the production of the circular segments it is ensured that the concrete cover will be built in very precisely. It is supposed that reinforcement can be embedded in a $x_c \geq 35$ mm concrete cover, assuming precise fabrication and attention to quality assurance measures. The geometrical variable concrete cover is therefore introduced as a exponential distribution with a relatively low standard deviation.

4.4.2 Chloride migration coefficient: $D_{RCM,0}$

For design purposes an important starting measure is the material resistance when focusing on the expected deterioration processes. Suitable results may occasionally be drawn out of the literature, which might be used as starting parameters in a service life design calculation. Oftentimes one can not draw quantitative results out of the literature, when working with special concrete mixes with for example very low water binder ratios and high contents of plasticiser. Therefore, it is essential to determine the efficiency of the materials in use by testing them in basic tests. This is to identify the suitability of the designed concrete mix. In this context the decisive material resistance is the chloride diffusion coefficient.

The determination of chloride diffusion coefficients is a rather complex and time-consuming procedure (Gehlen and Ludwig 1998). In practice, chloride penetration into concrete structures follows a non-steady state process. For this process, conventional methods are to immerse the concrete specimens in chloride contaminated solutions with constant chloride concentration. After immersion, the chloride profile is measured by sampling the specimen successively from the exposed surface and by analysing the total chloride content in each sample. The chloride diffusion coefficient can be found from the Fick's second law by curve fitting. In order to prepare a meso-level durability design in time before the structural elements are constructed, the measured material performance should be known as soon as possible. Among different rapid test methods, the rapid chloride migration method (RCM) revealed to be theoretically the clearest, experimentally the most simple and related to precision (repeatability) the most promising tool (Gehlen and Ludwig 1998). Diffusion data measured with conventional immersion methods correspond well to data determined with the rapid chloride migration method (Gehlen and Ludwig 1998; Tang 1996). How the measurement has to be undertaken can be seen in Gehlen and Ludwig (1998) and Tang (1996). The material under investigation gave values as provided in Table 3 as mean and standard deviation.

In addition to the measured performance at the time t_0 one has to consider the sometimes substantial difference in the time dependent development of the material resistance (Siemes *et al.* 1999; Bamforth 1998). The increase of the material resistance, or expressed by the opposite, the reduction of the chloride diffusion coefficient, will be expressed by the age exponent n (Variable 4 of Table 3). Typical quantities of n are given in Siemes *et al.* (1999) and Bamforth (1998).

4.4.3 Critical corrosion inducing chloride content: C_{crit}

In the Design Guide of the Comité Euro-International du Béton (CEB) for durable reinforced concrete structures of 1989 (CEB 1989), the relationship between the critical chloride content depending on the moisture content of concrete and on the quality of the concrete cover has been illustrated (Figure 4).

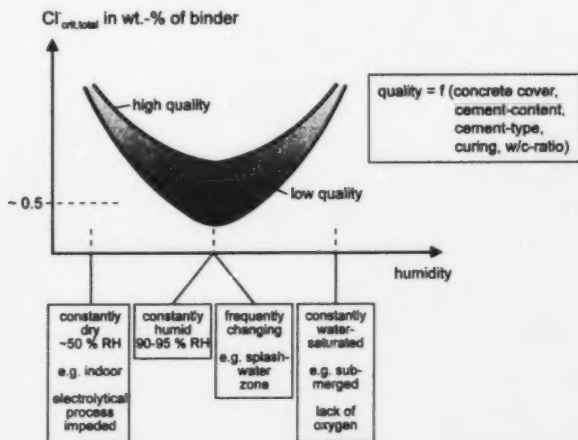


Fig. 4: Relationship between the critical chloride content to environmental conditions and quality of concrete cover (CEB 1989)

Whereas for permanent dry or permanent water saturated concrete practically no re-bar corrosion is possible, and therefore higher values for the critical chloride content can be admitted, appears the lowest value for the critical chloride content under permanent moist or alternating environmental conditions. In the case of a large concrete cover and a good concrete quality, which will be basically obtained by a low water/binder ratio and a suitable curing of the concrete, as well as through the use of additives, one can achieve higher critical chloride contents as compared to a insufficient quality of the concrete cover. The lowest critical threshold value for the chloride content was determined by Breit (1997) for various mortar mixtures. The surveys of Breit allow for the first time to introduce the critical corrosion inducing chloride content as a stochastic variable. An adapting test for the obtained results has shown that a normal distribution ($\mu = 0.48$, $\sigma = 0.15$) describes the results in a sufficiently exact manner.

In the surveys of Breit a set of different mixtures was investigated. The concrete cover had a value of only $x_c = 7.5$ mm. Making use of only one mixture and taking into account higher concrete covers, one can therefore estimate higher averages (due to increased concrete cover) and lower standard deviations (only one mixture). All the explained relationships justify the above mentioned assumption

(Table 3, Variable 3).

4.5 Result

With the arranged data it is now possible to calculate whether the stochastic variable x_c , which was applied in the design (Table 3) is sufficient to proof a serviceability limit state related minimum service life of $T = 100$ years. The required statistical calculation was carried out with the software package STRUREL (RCP GmbH: STRUREL 1995). The result of the evaluation is shown in Figure 5.

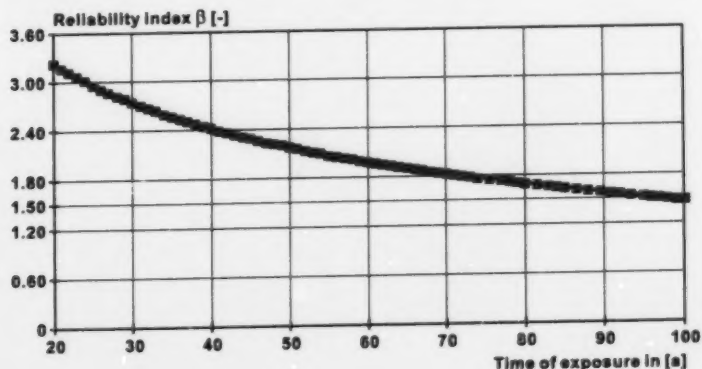


Fig. 5: Reliability index versus time of exposure (SLS: onset of corrosion)

Figure 5 shows, that with the tested material and the planned concrete cover one can achieve a durable structure concerning chloride induced steel corrosion with the minimum required safety according to the fixed serviceability limit state 'Onset of corrosion'. The obtained reliability index is $\beta_{SLS} = 1.50$ ($p_r = 0.068$), in accordance to the minimum required value of Table 2 ($\beta_{0,SLS} = 1.50$).

5 Additional measures

Beyond the basic testing (laboratory test), the quality level achieved in the laboratory should be reached on the construction site as well, since the quality of the built-in material decides over the service life of the construction. Criteria for the final examination (acceptance criteria) of the tested material resistances can be determined following EC1, appendix D. The continuous examination of the material variable $D_{RCM,0}$ cannot be undertaken on the construction site without significant effort, because for the construction site this method is too complicated to perform. Extensive surveys at the Institute for Material Research at the Technical University of Aachen (ibac) have shown a good correlation between the chloride migration coefficient $D_{RCM,0}$ and the electrolytic resistivity of concrete $\rho_{WER,0}$ (Figure 6)

undertaken with the so-called Wenner-probe. How to execute the Wenner method is described in detail in Gehlen and Ludwig (1998). Considering this aspect, the material resistance towards a chloride penetration was verified for the tunnel segments on concrete cubes stored submerged on the construction site by an indirect examination of electrolytic resistivity of the concrete.

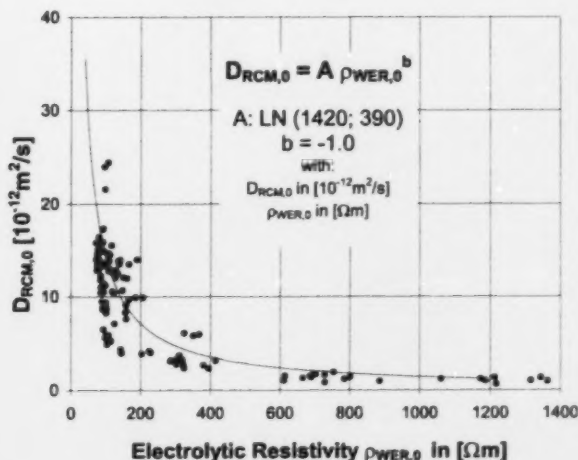


Fig. 6: Relationship between $D_{RCM,0}$ and $\rho_{WER,0}$

Further it has to be assured by a permanent measurement of the concrete cover, that the measured performance with regard to x_c is equal to the required performance $x_{c,r}$. Finally, other additional measures like hydrophobation in areas of restricted covers can be considered. By consideration of partly applied hydrophobation within the joints, the re-calculated reliability index at time $t = 100$ years is $\beta_{SLS} = 2.25$ ($p_f = 0.012$), higher than the required value of $\beta_{0,SLS} = 1.50 - 1.80$.

6 Concluding remarks

In the last decade much effort has been spent on the service life design of structures and especially of concrete structures. This has finally led to a situation where it was possible to make a fully performance and reliability based service life design for a real structure: the Western Scheldt Tunnel.

During the design, simplifications have been made. For example the target reliability indexes have directly been adopted from the structural design, both for the SLS and the ULS. For the ULS this is more or less obvious, as exceeding this limit

state will lead both for the structural design and the service life design to the same situation, namely loss of the structure. For some of the SLS's this is less clear. For example the onset of corrosion or the corrosion induced spalling are limit states that are related to the risk for repairs. This is primarily an economic issue and an economic optimisation of the reliability index is obvious. The optimisation is hardly possible because the amount of repair and the locations for the repair are difficult to estimate.

The service life design has been based on the reliability during the design service life. In other words it has been based on the total failure probability during the service life. Due to the degradation we have a situation where the failure probability grows with time. The failure rate (failure probability per year) in the first years is therefore low and in the last years relatively high. An improvement can be made by also restricting the failure rate.

The service life design has been made for the whole tunnel lining without differentiating the exposure zones. Especially for long tunnels with a high number of similar concrete elements this may lead to the acceptance that a high number of elements will fail. To restrict this the exposure zones can be differentiated or the target reliability index can be related to the amount of possible damage. For a small amount of failing elements a low reliability index can be applied. For a high amount of failing elements a high reliability index should be applied.

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2 DEGRADATION MECHANISMS AND SERVICE LIFE OF CONCRETE SLABS OF COMPOSITE BRIDGES

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Abstract

Composite steel-concrete constructions are presently widely used, and certainly in the field of composite bridges where they appear quite competitive. Most of the structural problems related to this type of construction are presently solved, concerning particularly the design of the steel girders. Nowadays studies are focused on the durability of the concrete slab, which conditions the performance of these bridges during their full service life. The model commonly accepted for the description of the corrosion process in a reinforced concrete element is presented. The evolution regarding the problem of the influence of the crack width on the durability of concrete is discussed. The factors leading to cracking of the concrete slab are examined, with special attention to thermal and autogenous shrinkage involving early cracking, and the results of a practical example are presented. The most essential requirements regarding durability are mentioned. Concerning reinforcement of the slab the classical solution consists in using passive steel. However, as the slab is cracked, durability will be controlled by the corrosion development, which leads to uncertainty regarding service life. A satisfactory performance during a sufficiently long period can be ensured by prestressing. Several parameters have to be assessed carefully, such as the type of prestressing and the amount of prestress to be introduced in the slab.

Keywords : bridges, composite steel-concrete construction, concrete quality, corrosion development, cracking, passive and active reinforcement, serviceability limit states.

1 Introduction

During the last decades the use of composite steel-concrete construction in bridge design has shown considerable extension, which can be explained by several developments and innovation techniques (Johnson and Buckby 1988). The material characteristics have been improved for steel and for concrete. A better understanding of the structural behaviour, as well as improvements of detailing and construction methods must be pointed out.

Presently the innovations that have to be brought to bridge construction, and more generally to structural design are related to various criterions such as cost, detailing, durability, aesthetics and environmental conditions. In this article, only the problem of durability will be examined and more particularly the durability of the concrete slab, which cannot be ensured easily, as the concrete deck of a bridge is submitted to severe environmental conditions.

The slab can be reinforced by passive or active steel. When passive reinforcement only is used, cracks will open and there is some doubt about durability conditions for a very long period. In Belgium large span composite bridges have been built 20 years ago with passive reinforcement in the slab, and up to now very few in situ observations have been made regarding the degradation of the concrete deck. An investigation program will start in spring 1999 to examine the cracking situation in the slab of some of these bridges.

Prestressing of the slab is considered as a promising solution to ensure a satisfactory performance. Despite this there are still discussions going on in Belgium and in some European countries on whether or not it is advisable to use prestressing in future composite bridges, since in this case prestressing is introduced for durability and not for stability purposes.

The aim of this paper is first to show the importance of calculating precisely the tensile stresses and cracking development by considering the example of a classical continuous bridge. Another important feature is the economical level of prestress to be prescribed to the practitioners and design engineers. A discussion is presented on this matter and a proposal is made with respect to the various load combinations considered presently in the new European codes.

2 Evolution in the design of composite bridges

For large span bridges, the tendency is nowadays to use composite structures. Such structures take advantage of both the light weightness and the high strength of steel components, and of the low cost of the concrete slab.

Forty years ago steel structures were used for large span bridges, with plate or box steel girders and an orthotropic steel deck plate. Because the construction of such a deck is complicated and requires skilled labour, and because the cost of manpower increases continuously, this type of bridge has become extremely expensive. Under

the pressure of competition, a concrete slab has been progressively substituted for the orthotropic steel deck plate.

Developments in composite bridge design have been accompanied by theoretical and experimental research studies (Lebet 1988). The results of these studies have been incorporated in national and international recommendations (Eurocode 4 - Part 2: Bridges 1997).

The design of composite bridges must meet the two classical requirements: verification of ultimate limit states and of serviceability limit states. Regarding serviceability limit states particular attention is paid nowadays to the durability of the concrete slab. Cracking of the slab is not the only parameter to be considered, but it is one of the most important.

The majority of composite bridges have continuous spans. It is common to find bridges with ten or more spans with typical lengths varying between 30 and 90 m. Looking at the classical bending moment diagram for a continuous beam (Fig 1), it can be seen that hogging moments exist near the supports. Therefore tensile stresses should appear in the concrete slab only in these regions. This is not true for composite bridges due to various factors (creep, shrinkage, external temperature variation) that will be examined later on. Therefore tensile stresses and cracks may develop everywhere in the slab, near the supports but also at mid-span.



Fig. 1 : Bending moment diagram in a continuous beam

3 Deterioration mechanisms of the concrete slab

3.1 Short considerations on the service life of bridges

Ageing and deterioration of bridge structures are unavoidable. Therefore the main task must be to control the types and rates of deterioration. A durable construction implies that this construction does not necessitate important rehabilitation and renovation works.

The service life of bridges must be specified. It is presently agreed that this period of time is situated between 50 and 100 years, and close to 100 years. A duration of 50 years is too short and non economical. For most bridges a duration of 100 years seems a little bit too long due to the evolution of the traffic needs, and 80

years is presently considered as an optimum service life. In case of exceptional structures, like the Oresund and Vasco da Gama Bridges, a service life of 100 years or more may be required, due to the difficulty and cost of rehabilitation works.

3.2 Corrosion process in a reinforced concrete element

Two dominant factors are involved in most chemical and physical processes influencing the durability of concrete structures: transport within the pores and cracks, and free water. More particularly, in the case of the concrete deck of a bridge the following factors have to be considered: freezing and thawing cycles, effects of de-icing salts, penetration of chemically aggressive agents, alkali-silica reaction (Bulletin d'Information N°183 of CEB 1992). It appears therefore that appropriate durability cannot be ensured easily, as the concrete slab is submitted to severe environmental conditions.

Corrosion is the most critical degradation process, as it can lead to failure of the element. Therefore the mechanisms of corrosion must be examined carefully. Corrosion is mainly due to carbonation of concrete in relation with penetration of CO_2 , and to penetration of chloride ions originating from de-icing salts.

Passivation of steel is due to the alkalinity of concrete. In such an environment the corrosion rate is extremely low and steel is therefore protected. The passivity of steel may be destroyed by the carbonation of concrete surrounding the reinforcing bars and by the penetration of chlorides through the pores. For passive or active reinforcement situated near the top or the centre of the concrete slab of a composite bridge, the penetration of the chloride ions is the prevailing action.

During the last two decades, designers and researchers have focused on the deterioration processes existing under different types of environment. As a result, rational and scientifically sound models of major deterioration mechanisms, and more particularly of the corrosion process, are available today (Schiessl 1988; Andrade et al. 1993; Bulletin d'Information N° 238 of CEB 1997; Liu and Weyers 1998).

On the basis of these theoretical and experimental studies, it has been shown that the corrosion mechanisms develop through two different phases as illustrated in Fig 2.

In the initiation phase, the metal embedded in concrete remains passive. No deterioration or damage can be seen, but some inherent resistance against the prevailing environment is diminishing or overcome by the aggressive media, e.g. penetration of chlorides or carbonation. These environmental changes lead ultimately to depassivation.

In the propagation phase, which begins at the moment of depassivation, corrosion develops, often at an accelerating rate, until a final stage is reached. The corresponding damage to the structure is only visible when this propagation phase has been reached.

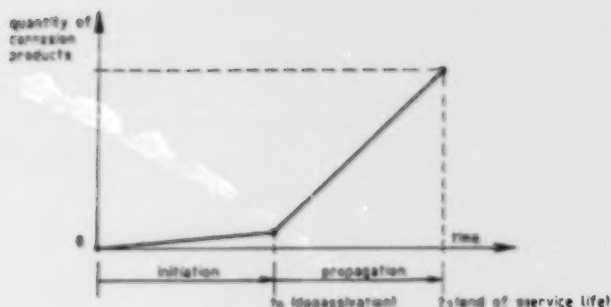


Fig. 2: Schematic representation of corrosion process

Corroded steel develops rust products which expand and cause cracking and deterioration of the concrete cover. A further reduction of the steel area may result in falling concrete fragments. Sudden failure may occur if longitudinal cracking along the bar develops in the region of the bar anchorage. These unacceptable damages usually correspond to the end of the service life of the element (time t_1 in Fig 2).

For the design engineer two possibilities can be envisaged:

- $t_0 >$ expected service life: this solution is quite safe, since any depassivation of steel is avoided;
- $t_1 >$ expected service life: the safety level is not known precisely, as it is difficult to assess the propagation period due to the number of parameters involved

3.3 Influence of crack widths on durability of concrete

Twenty years ago it was admitted that crack widths had a significant effect on the corrosion process. There was then a strong gradation of the admissible crack widths in the existing recommendations. There is now a considerable evolution regarding this problem. The observation of existing constructions and laboratory tests have shown that there is no direct relation between the crack widths and the degree of corrosion provided they remain smaller than 0.4 mm approximately (CEB-FIP Model Code 90 1993; Favre et al. 1997).

However the existence of cracks, even with a small width (0.1 mm), does influence significantly the corrosion process. The diffusion of chloride ions is ten times more rapid in a cracked than in an uncracked concrete (Schiessl 1988). This means that the initiation period will be approximately ten times longer in an uncracked material compared with a cracked one, provided that in both cases the permeability of the material is low and concrete cover is sufficient. Therefore, in an uncracked concrete, t_0 will be large enough to prevent the steel from reaching the propagation

stage during the service life. If concrete is cracked, t_0 is small with respect to the service life, and t_1 becomes the critical parameter with much more uncertainty regarding structural safety.

4 Development of cracking in the concrete slab

4.1 Early cracking

Cracking before hardening is due to plastic settlement and plastic shrinkage. These two phenomenons may induce important cracking, but preventive measures can be adopted in order to avoid them, and it will not be examined here.

Cracking occurring just after hardening is mostly due to two phenomenons: thermal shrinkage and autogenous shrinkage.

The behaviour of concrete during hardening is important in elements with large thicknesses like dams and bridge piles. For elements with moderate thicknesses (20-30 cm), the temperature differences between the edges and the centre are small. However, if the displacements of concrete during the heating and cooling phases are restrained by another structural element, self-equilibrated stresses appear and may induce early cracking (Emborg 1989). In composite bridges the steel profile is the restraint element, when connection is ensured between concrete and steel during concrete hardening.

The tensile stresses related to thermal shrinkage are due to the fact that the modulus of elasticity varies considerably between the heating and the cooling phases. Research performed in this domain has shown that the main parameter to be considered is the ratio between the area of the steel profile and the area of the concrete slab (Emborg 1989).

Another factor often neglected is the effect of autogenous shrinkage, i.e. shrinkage due to self-dessication. Since its magnitude is of the order of 10^{-4} for ordinary concrete ($w/c > 0.45$), it has been ignored for practical purposes. However concrete with low water-cement ratio is used more and more extensively. In this case autogenous shrinkage increases very rapidly, and it can reach 2 to $3 \cdot 10^{-4}$ when $w/c < 0.40$, which can lead to non-negligible tensile stresses and early cracking (Tazawa and Miyazawa 1993).

4.2 Cracking under service conditions

There exists at least four causes of cracking under service conditions: drying shrinkage, creep, external loads, external temperature variations (daily and seasonal).

Drying shrinkage has the same effect as thermal and autogenous shrinkage, but it develops for a much longer period.

Creep does not create tensile stresses, but as it cannot develop freely due to the steel profile, it induces relaxation of the compressive stresses in the concrete slab.

External loads such as truck loading may lead to various types of stresses in the deck according to their position along the bridge (Lebet 1990).

External temperature variations (daily and seasonal) induce general thermal effects in hyperstatic structures and self-equilibrated stresses in the concrete slab. The determination of these effects is rather involved (Cope 1987). The temperature distribution on a composite cross-section is slightly different from that on a full concrete cross-section.

4.3 Example of a classical continuous bridge

In order to evaluate the tensile stresses and cracking development that may occur in the slab, the example of a classical composite continuous bridge with two spans has been analysed (Fig. 3). The stress distribution has been calculated precisely using the computer code SAFIR developed at the University of Liège. Values recommended for external temperature variations have been adopted (Cope 1987).

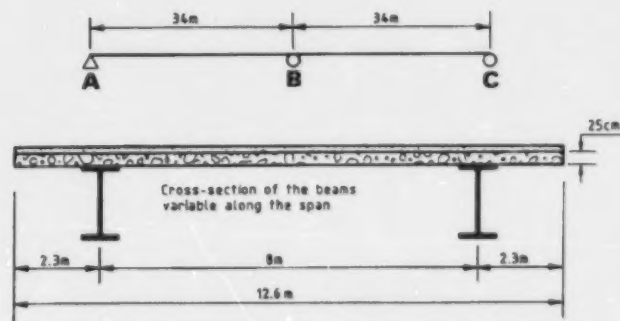


Fig. 3: Classical composite continuous bridge with two spans

Table 1: Tensile stresses in the concrete slab under service conditions

Position in the beam	Top stress (MPa)		Bottom stress (MPa)	
	perm	perm + var	perm	perm + var
Mid-span	1.2 (1)	2.5 (2)	1.7 (1)	2.2 (2)
Support	5.1 (1)	7.8 (3)	4.1 (1)	5.3 (3)

(1) : permanent loads + early shrinkage + drying shrinkage

(2) : (1) + external temperature variation

(3) : (1) + variable loads (rare combination) + external temperature variation

Table 1 shows the maximum tensile stresses under permanent and permanent + variable conditions. It can be seen that tensile stresses are present in the slab during the whole life even at mid-span and that cracking may occur both near supports and at mid-span.

5 Control of cracking

5.1 Essential measures

A few essential measures are required in order to obtain adequate durability of the concrete slab of a composite bridge:

- to place a waterproof membrane of good quality.
- to use a concrete with good mechanical characteristics and very low permeability.
- to provide sufficient concrete cover.
- to adopt an appropriate concreting sequence for slabs directly cast in situ on the steel structure.

5.2 Passive reinforcement

The classical solution is the use of passive reinforcement. In this case cracks will open and it is impossible to limit their width to values < 0.1 mm. As mentioned previously, it is now acknowledged that crack widths up to 0.4 mm do not influence the corrosion development. However adequate detailing regarding the reinforcing bars should be recommended in order to limit the crack width to 0.15 or 0.2 mm. Nowadays many bridges are subjected to very heavy traffic loads. Crack widths may tend to increase, due to progressive deterioration of bond at the boundaries of the cracks. Additional research studies on this matter and observations on existing bridges should be performed.

The limitation of crack width can be reached by appropriate measures:

- a minimum amount of passive reinforcement.
- a limitation of the tensile stresses in the rebars.
- a careful study of the spacing between the bars.
- a limitation of the diameter of the bars.

It must be pointed out that bars with small diameters have also some drawbacks. They are more susceptible to corrosion. Furthermore concreting may become more difficult due to the density of steel, which may lead to a diminution of concrete quality.

De-icing salts will cause chloride ions to penetrate in concrete. As the slab is cracked, the rate of penetration is very high, depassivation occurs quickly and the durability is controlled by the corrosion development. In these circumstances it is presently not possible to ensure a service life of 80 years. Values situated between 20 and 40 years are sometimes mentioned, but this has to be confirmed by additional research studies.

5.3 Prestressing

In order to improve the durability of the slab prestressing can be used. Transversal and longitudinal prestressing have both beneficial effects. In this article only longitudinal prestressing will be considered. Several factors have to be examined carefully.

The project must remain economical, as prestressing is introduced for durability purposes only and is not necessary regarding structural behaviour. It is furthermore difficult to calculate the stresses induced by prestressing of the slab. The efficiency of prestressing is reduced by the composite interaction and by the classical time-dependent losses due to creep, shrinkage and relaxation.

The type of prestressing must also be examined carefully. Several procedures are used and the various methods have their advantages and drawbacks.

1. Prestressing by jacking supports. In this method the steelwork is raised above its final level before the slab is cast. After curing of the slab the composite section is lowered to its final level. In continuous composite bridges with large spans the required vertical displacement of the steel beams becomes very large and furthermore, with this method, prestressing losses in the concrete slab may be as high as 50 %.
2. Prestressing the slab and steel section (entire cross-section) using cables placed longitudinally in the concrete slab. This method is often used simply to prestress regions near internal supports. Large cracks can occur near anchorages if adequate reinforcement is not provided. Furthermore, if the slab is prestressed over the entire bridge length, losses may approach 50 %.
3. Prestressing the slab only, using cables placed longitudinally in the slab. In this method, the slab is prestressed before being connected with the steelwork. This method is convenient for precast slabs. It is also suitable when the slab is cast in situ if holes are left in the slab around groups of shear connectors during concreting. During prestressing, the slab slips on the upper flange of the steel structure. In this procedure prestressing forces and losses are smaller than those observed when prestressing the entire composite section. However it is difficult to achieve a satisfactory shear connection.
4. Prestressing with external cables using truss action. This method is sometimes used to strengthen existing bridges. In this system cable control and maintenance are facilitated, and replacement and addition of cables may be performed easily. The loss of prestressing due to creep and shrinkage is also minimised. Disadvantages are costly anchorages and cable supports, and application of concentrated forces to the steel profile.

It is important to define the amount of prestress to be introduced in the slab in order to ensure sufficient durability for a service life of approximately 80 years. This question has so far not received enough attention. It is examined here with respect to the load combinations now defined in the European recommendations (Eurocode 1 – Part 1 1995).

First of all the initial prestress to be considered has to take into account prestressing losses due to creep and shrinkage. Therefore, for large span bridges, procedures 1 and 2 mentioned previously should not be recommended, since prestressing losses in the concrete slab are very high.

Three combinations of actions for serviceability limit states are defined : quasi-permanent, frequent and rare.

The quasi-permanent combination for a bridge includes no traffic loads and no external temperature variation. In this case, prestress must be such that under actions existing at any time, i.e. permanent loads including early shrinkage, drying shrinkage and creep, no crack would occur. In this situation cracks will open under variable actions such as traffic loads and variation of external temperature. This type of design can be unsafe as cracks may be present during rather long periods and heavy traffic loads may cause fatigue effects. Recent research studies have shown that crack widths increase with increasing number of load cycles or with time under sustained loading (Balazs 1993 ; Bulletin d'Information N° 235 of CEB 1997). Consequently a progressive increase of the crack widths may occur and after a certain time some cracks may remain permanently open.

Another combination includes quasi-permanent actions and external temperature variation. As it is not critical, it will not be discussed here.

The highest level of prestress is to be applied for the rare combination of actions including external temperature variation. This combination has a return period of more than one year. In this case cracks will open very rarely in limited areas of the concrete deck. This design is suitable for durability purposes, but it is very difficult to introduce such a high level of prestress for technical and economical reasons. In the example described in Fig.2 and Table 1, it has been calculated that a compressive stress of approximately 5.5 MPa should be introduced in the slab in order to fulfil this condition.

It is more appropriate to define the level of prestress from the frequent combination of actions including external temperature variation. This combination has a return period of a few days. This appears to be a good compromise between durability and economy. Referring to the same example a compressive stress of approximately 4 MPa should be introduced, which is technically much more acceptable.

6 Conclusions

The following conclusions can be drawn from this research study.

1. A few essential measures are required in order to obtain adequate durability of the concrete slab of a composite bridge.
2. The causes of cracking have been analysed.
With the use of concrete with low w/c ratio, early cracking due to thermal shrinkage and autogenous shrinkage should be considered carefully. Cracking

under service conditions has also been examined. It has been shown that cracking may exist during the whole service life of the structure, not only on the supports but also at mid-span.

3. If concrete is cracked depassivation of reinforcing steel will arrive rather quickly and durability will be controlled by the corrosion development. This leads to uncertainty regarding service life.
4. The classical solution is the use of passive reinforcement. Despite the recent studies on the influence of crack widths, adequate detailing regarding the reinforcing bars should be recommended, as the crack widths tend to increase due to heavy traffic loads inducing fatigue effects. Additional research studies and observations on existing bridges should be performed.
5. In order to improve the durability of the slab prestressing can be used. The example of a classical composite continuous bridge has shown the various loads and effects that have to be taken into account, and the way of combining them to calculate precisely the tensile stresses appearing in the slab. Full prestressing will lead to very high compressive stresses and is not economical. On the other hand, to allow crack development for all variable loads may be unsafe due to fatigue effects. Therefore on the basis of the load combinations defined presently in the European codes, it is proposed, in this article, that the frequent combination would be the reference for the amount of prestress to be considered.

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3 SERVICE LIFE AND SAFETY PREDICTION OF CONCRETE SLABS AT FILLING STATIONS

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Abstract

In Germany there are almost 19.000 filling stations. Water-pollution-prevention-regulations have become stricter and in order to fulfil the new demands the bulk majority of filling stations have to renew their tanking area. Modest approximations render the tanking area of a filling station has a common size of 200 m² and the renewing costs of tanking area being DM 200/m². So the total tanking area which has to fulfil the water-pollution-prevention-regulations comes to 3,8km² and the total costs comes to 760 Mio. DM. Thus the life expectancy of concrete slabs in terms of tightness is a matter of environmental and economical importance. In-situ tests gave the mean depth of penetration at filling stations after averagely 5 years of service less than 40mm. None of the measured penetrations was larger than 1/3 of the existing slab thickness. The penetration at concrete/joint is not larger than in an area of concrete only. Between porosity and penetration there is a clear relationship and this relationship is stronger than of tanking operations or age of lane. Based on those tests, reference tests in laboratory and calculations the life expectancy was evaluated and found to exceed 30 years.

Keywords: Life expectancy, penetration of fuel in concrete, filling stations, concrete slabs

1 Introduction

The penetration of a fluid medium into a porous Medium is a complex function with many dependent factors. In a highly porous substrate the process of penetration can fairly ease be described with the well known law of Darcy, e.g. groundwater in gravel. The fluid penetration into concrete, where the bulk majority of the porous

space is capillary pores the law of Darcy do not apply as sole function (for capillary pores the influence of hydraulic head less than 1.5m be neglected) and the capillary process becomes dependent of the capillary movement. An extensive research in Germany (DafStb 1996), have rendered the following function for the continuous penetration process of organic fluid:

$$e_t = \sqrt{\frac{\cos \theta \cdot r}{2}} \cdot \sqrt{\frac{\sigma}{\eta}} \cdot \sqrt{t} \quad (1)$$

e_t	penetration at time t	(m)
σ	surface tension	(N/m)
η	dynamic viscosity	(N s/m)
θ	contact angel ($\cos \theta$)	(-)
r	capillary radius	(m)
t	penetration time	(s)

It can easily be shown that when penetration e_t at time t is known and assuming the same fluid and concrete (σ , η , θ and r constant) the penetration e_{t1} at another time t_1 can be extrapolated by:

$$e_{t1} = e_t \cdot (t_1 / t)^{1/2} \quad (2)$$

At filling stations the penetration process is not continuous but intermittent due to possible exposure caused by spilling of fuel during the tanking operation. Some amount of the fuel penetrates and some evaporate. Thus the intermittent penetration additionally becomes dependent to spilling amount (human behaviour, tanking frequency) and the vapour pressure of the exposed fluid.

In a cooperation between the German Society for Petroleum and Coal Science and Technology (DGMK), and TU-Darmstadt tests and calculations were carried out (DGMK *et al.* 1996).

The aims of this cooperation were:

- to render a state of the art regarding the penetration of fuels at filling stations
- establish a calculation basis for the intermittent penetration process
- evaluate the life expectancy of the concrete slabs in the filling area

The evaluation of the life expectancy was based on the DafStb - Guideline (DafStb 1996). In this guideline, the term tightness is defined in the following way: The penetration front of the fluidal medium does not provably reach (with a margin of safety) the action-opposite side of the concrete structure during its time of action.

Hence uncracked concrete structures can be regarded tight when the following criteria is fulfilled:

$$d \geq e_{tk} \cdot \gamma_c$$

- where:
- d : thickness of concrete structure
 - e_{tk} : characteristics depth of penetration ($e_{tk} = 1,35 \cdot e_{t,m}$)
 - $e_{t,m}$: mean depth of penetration, test result
 - γ_c : safety factor for penetration

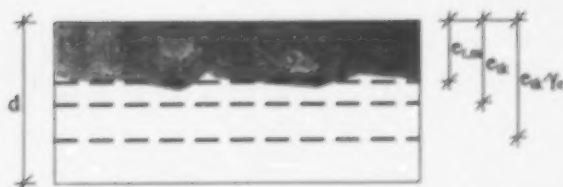


Fig. 1: Definition of tightness according to DAfStb-Guideline (DafStb 1996)

2 Results

2.1 In-situ test results

The in-situ tests were carried out at 9 filling stations numbered T1-T9. Relevant station data is shown in Table 1.

The width of the joints was between 13 and 20 mm. The depth was between 11 and 25 mm. The mean depth of penetration was measured from borehole cores taken from only concrete and the concrete at the joints of slabs. The porosity was also measured and the results for each of the filling stations are given in Table 2.

The following relationships to penetration were investigated:

- porosity
- number of tanking operations
- age of lane

The penetration as a function of tanking porosity is shown in Figure 2.

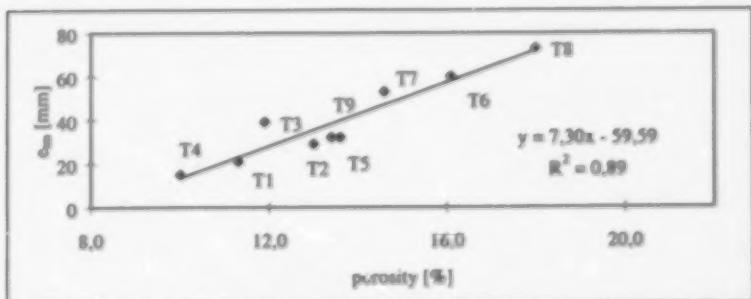


Fig. 2: Penetration as a function of tanking operations

From Figure 2 a clear correlation between penetration and number of tanking operations can be seen.

Table 1: Data on tested filling stations

No.	Location	Slab size [m-m]	Slab thick., d [cm]	Reinforcement	Petrol sold [l/month]	Diesel sold [l/month]	Service Start date	Test date	Age of slab [months]
T1	country	8.1 x 8.4	20	Q257 U	50 000	60 000	10/89	9/95	71
T2	highway	5.9 x 3.7	20	no inform.	non	130 000	5/90	9/95	64
T3	city	7.4 x 6.4	13-20	Q188 O+U	75 000	25 000	10/90	2/96	64
T4	city	4.4 x 4.5	20	Q188 O+U	97 000	15 000	11/90	2/96	63
T5	city	4.8 x 5.9	18	Q188 O+U	non	30 000	6/91	2/96	56
T6	city	5.0 x 4.2	24	2Q188 O	120 000	30 000	7/92	3/96	44
T7	city	5.4 x 3.7	21-25	Q188 O+U	non	150 000	6/92	3/96	45
T8	city	3.5 x 3.0	21	Q188 M	non	40 000	11/91	3/96	52
T9	city	3.2 x 3.0	18-22	non	non	70 000	8/91	4/96	56

Table 2: Mean depth of penetration e_{cm} and porosity (in-situ)

Filling station	Mean depth of penetration e_{cm} [mm]		Total porosity [%]
	concrete	concrete/joint	
T1	21	no value measured	11.3
T2	29	no value measured	13.0
T3	39	no value measured	11.9
T4	15	no value measured	10.0
T5	32	30	13.4
T6	60	50	16.1
T7	53	50	14.6
T8	73	no value measured	18.0
T9	32	32	13.6
mean $\bar{e}_c$	39	40	
Standard	19	11	

To evaluate the effective number of tanking operations the following assumptions were made:

the average tanking amount per tanking operation is 40 litres (DGMK *et al.* 1996)
for each tanking operation the spilled amount is 4 ml (Scott Research Laboratories Inc. (1972))

Additionally, tests made in Darmstadt have shown that the penetration of petroleum being about 0.16 times the penetration due to diesel oil. This can be explained through the higher evaporation rate of petroleum. Based on those assumptions and the consumption data in Table 1, the effective number of tanking operations from the service start date to the test date was estimated as provided in Table 3.

Table 3: Number of tanking operations

Filling station	Tanking operations [10^3]
T 1	121
T 2	208
T 3	59
T 4	48
T 5	42
T 6	54
T 7	169
T 8	52
T 9	98

The penetration as a function of tanking operations is shown in Figure 3.

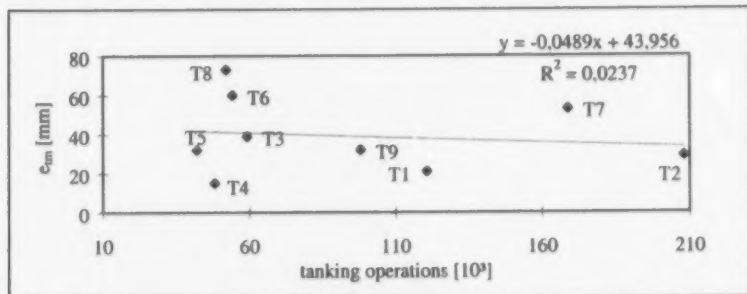


Fig. 3: Penetration as a function of tanking operations

It is evident from Figure 3 that no correlation exists between penetration and number of tanking operations can be seen.

The penetration as a function of tanking operations is shown in Figure 4.

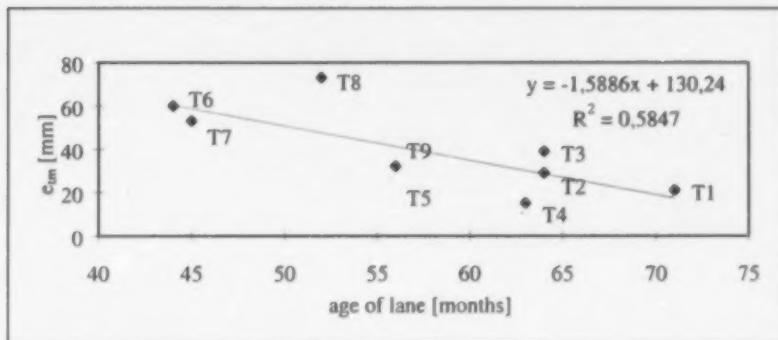


Fig 4: Penetration as a function of age of lane

From Figure 4, a weak negative correlation between penetration and the number of tanking operations can be observed.

Hence, based on the results provided in Figure 2-4, the following can be said:

- The mean depth of penetration at filling stations after an average of 5 years of service is less than 40mm.
- None of the measured penetrations was larger than 1/3 of the existing slab thickness.
- The penetration at the concrete/joint interface is not greater than in a given area of concrete only.
- There is a clear relationship that exists between porosity and penetration and this relationship is more significant than that of tanking operations or age of lane.

2.2 Laboratory results

A concrete was mixed, cured and acted upon intermittently according (DafStb 1996). Also an uncontaminated piece of concrete was taken from filling station T1 and loaded identically. Thus a comparison between the in-situ tests and laboratory action was possible. The results are shown in Table 4.

Table 4: Mean depth of penetration e_{LM} (laboratory)

Duration and type of fluid action	Depth of penetration e_{LM} [mm]					
	28 days each 5 hours ($\Sigma 140$ hours)		144 hours continuous		6 years tanking spillage	
Test-Medium	diesel oil		diesel oil		diesel oil/ petroleum	
Test area	concrete	concrete/joint	concrete	concrete/joint	concrete	concrete/joint
Laboratory concrete	12	15	14	13	-	-
In-situ concrete T1 (no pre spillage)	20	13	15	13	-	-
In-situ concrete T1 (max. spillage)	-	-	-	-	21	-

Based on the results in Table 4 the following can be said:

The mean penetration resulting from the laboratory action according to (DafStb 1996) is comparable to the penetration resulting from 6 years at filling station T1.

The penetration values at concrete joints do not exceed those for continuous concrete area.

3 Life expectancy and conclusions

The assumption was made that all filling stations tested have similar environmental conditions (weather). Further tests (Gunnarsson, S. 1998) have shown that the fuels exposed to the concrete slabs are comparative. It has been shown that there is a clear relation between the porosity of the concrete and the intermittent penetration process. Using the data achieved in the in-situ and laboratory tests and the penetration model in (DafStb 1996), the life expectancy was evaluated in two ways:

1. using the 5th percentiles of the penetration and the penetration model from (DafStb 1996)
2. using the penetration with a safety value of $\gamma_c = 1.5$ and the penetration model from (DafStb 1996)

The evaluation and comparison of those two different approaches is provided in the following:

Method 1. 5th percentiles of the penetration and the penetration model from (DafStb 1996)

Meanvalue $\bar{e}_{l,m} = 39$ mm; standard deviation, $\sigma = 19$ mm; number of stations $n = 9$

5th percentiles: $e_k^{95} = \bar{e}_k + k \cdot \sigma = 39 \text{ mm} + 2,193 \cdot 19 \text{ mm}$
 $= 80 \text{ mm} < 200 \text{ mm}$ (thickness of the slab)

Using the model from (DafStb 1996) gives:

$$e_{l,m} = e_{t1,m} \cdot (t/t_1)^{1/2} \quad \text{with: } e_{t1,m} = 39 \text{ mm and } t_1 = 5a$$

So the life expectancy, time = t , for a slab thickness $e_m = d = 20$ cm:

$$t = (d / e_{t1}^{5\alpha})^2 \cdot t_1 = (20 \text{ cm} / 8,0 \text{ cm})^2 \cdot 5 a = 6,3 \cdot 5 a = 32 a$$

Method 2 penetration with a safety value of $\gamma_e = 1.5$ and the penetration model from (DafStb 1996)

Using the penetration model and the permitted value of penetration from (DafStb 1996), then the life expectancy, time = t , for a concrete slab of = 20 cm:

$$t = (d / \gamma_e \cdot 1,35 \cdot e_{l,m})^2 \cdot t_1 = (20 \text{ cm} / 1,5 \cdot 1,35 \cdot 3,9 \text{ cm})^2 \cdot 5 a = 32 a$$

Both approaches render the same life expectancy of 32 years.

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4 ESTIMATION OF SERVICE LIFE OF CONCRETE PIPES IN SEWER NETWORKS

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Abstract

The expense of repair and maintenance of private and public sewage systems in Germany runs to about 100 billion US \$. This primarily affects concrete pipes and brick-work sewers. Approximately 40 per cent of the damage can be attributed to corrosion by biogenous sulfuric acid attack as a result of long flow periods and insufficient ventilation of waste water. The report deals with investigations into the prediction of concrete corrosion by biogenous sulfuric acid.

For testing of service life in cases of dissolute and expansive chemical attack a very simple easily reproducible comparative simulation method has been developed. The specimens are stored in diluted sulfuric acid for 35 days. After 6 days the specimens are removed from the solution, stored under running water for 24 hours and all loose parts removed in an ultrasonic vessel. This procedure corresponds to the practical conditions of cyclic flooding in combined sewers. After the exposure time the undamaged cross-section, the residual weight, the residual compressive strength and the acid penetration depth are determined.

The ageing progress of the specimens stored in acid is compared with the observed corrosion of concrete pipes in real sewer systems. The result is a very good correlation between the calculation of corrosion by Pomeroy's equation and the time-accelerated laboratory test involving storage in diluted sulfuric acid. The predicted corrosion after laboratory tests in sulfuric acid at pH value 2 corresponds to the average observed corrosion in a 20-years-old combined sewer.

Keywords: Concrete sewer pipe, time-accelerated test, biogenous sulfuric acid attack, durability, prediction of chemical corrosion

1 Introduction

The historical development of the German sewerage network shows that the majority of sewers are more than 70 years old and have reached the end of their service life. Of the pipe materials used, concrete and stoneware predominate for pipe dimensions of 200 to 1.200 mm. Most damage is caused by cracks, defective pipe connections, joint displacements and corrosion. The restoration of the damaged sewerage network, indispensable for ecological as well as economic reasons, is estimated to cost about 100 billion US \$. Figure 1 shows some technical data on the German sewerage network (Kaempfer 1998).

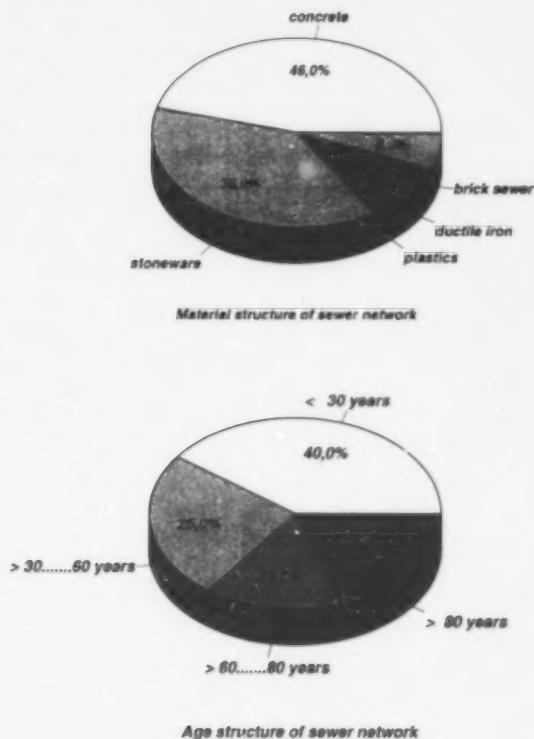


Fig. 1: Technical data on the sewer network in Germany

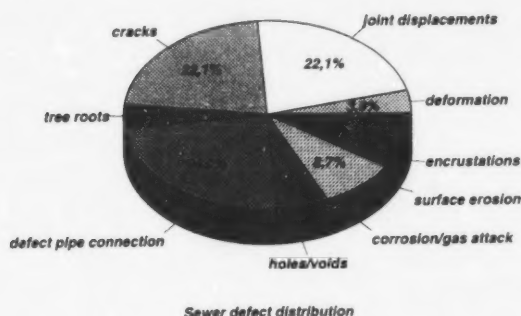


Fig. 1, cont'd: Technical data on the sewer network in Germany

Many sewers show pipe damage on account of

- improper construction
- improper sewage engineering and
- high traffic loads.

In a large number of cases, damage in sewer pipes will actually result in exfiltrations into ground water. It is estimated that, due to this damage, 500 million cubic meters of contaminated liquids per year can leak into soil and ground water. This quantity of waste water represents an overall risk for the environment and public health. Damaged sewer systems are one of the main sources of underground contamination with sulfate, chloride and nitrogen compounds.

Approximately 40 per cent of the damage in concrete and brickwork sewers can be attributed to biogenous sulfuric acid attack. The resistance of conventional mortar and concrete using standard mix composition is not sufficient to prevent chemical corrosion in the long flow of waste water (Kaempfer 1998).

2 Sulfide generation and corrosion

Under certain conditions hydrogen sulfide, which occurs particularly in sewerage, is converted into sulfuric acid by the action of sulfur bacteria. As a result, a sulfuric acid concentration of several percent may be reached in a matter of weeks. With regard to the intensity of the acid formation, the temperature and the flow period of waste water are very important factors in the formation of sulfuric acid. As the bacteria which produce sulfuric acid can grow only in the 0.5 to 6.0 pH range, a reduction in surface pH value is essential when cementitious material is attacked. This is facilitated by the presence of CO_2 in the atmosphere of sewerage canals (Meyer 1980).

The term biogenous sulfuric attack is used for processes in sewerage systems where microorganisms form sulfuric acid and cause the cement paste matrix to be attacked until it dissolves. Damage due to biogenous sulfuric acid leads to a scrubbed

concrete surface above the effluent line in part-flow sewer pipes, especially dirty waste water sewers. The typical appearance of a chemically-damaged pipe wall surface is shown in Figure 2.

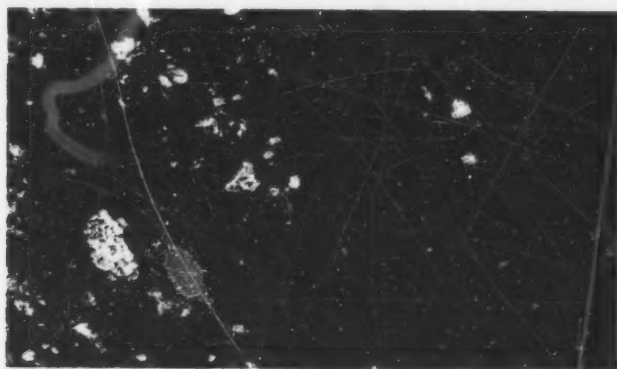


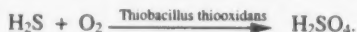
Fig. 2: Damage by biogenous sulfuric acid attack in sewer pipe atmosphere

The formation of harmful biogenous sulfuric acid depends on the presence of components in the sewer that contain sulfur. Insufficient ventilation of sewer pipes leads to the accumulation of hydrogen sulfide in the atmosphere on the pipe walls. This is then oxidized into elemental sulfur by the oxygen. The biochemical activities of differing microorganisms of the species *Thiobacillus* lead to the formation of sulfuric acid (Schmidt 1997).

The reduction of sulfate (SO_4^{-2}) in the presence of waste organic matter proceeds as follows



Sulfate-reducing bacteria thrive in the slime layer that coats the inner wet pipe wall. Oxygen cannot normally penetrate this layer, leading to the formation of an inert anaerobic zone next to the pipe wall where the sulfides are formed. Hydrogen sulfide in the atmosphere can be oxidized on moist surfaces by bacteria, producing sulfuric acid according to the following reaction (Meyer 1980):



This can lower the pH value on the surface to 1.0 or even lower and can cause a very strong acid attack. The corrosion rate of the sewer pipe wall is determined by the rate of sulfuric acid generation and the properties of the cementitious materials.

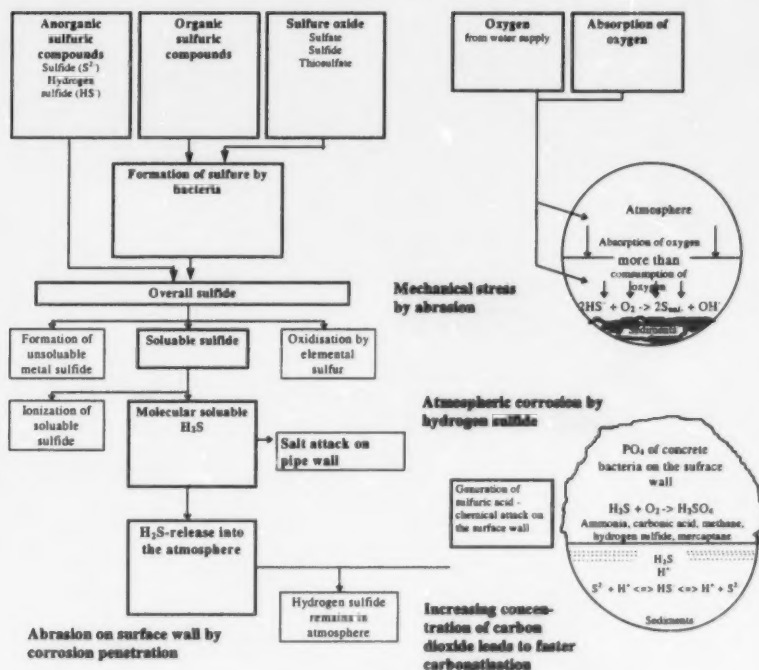


Fig. 3: Sulfide generation and corrosion in domestic and industrial waste water systems

The rate of corrosion of concrete pipes depends on the strength and density of concrete, the degree of acid penetration, the acid value and the circulation of hydrogen sulfide in the atmosphere. An important influence on the rate of corrosion is exerted by the geometry of the pipes and the filling capacity. The theoretical corrosion prediction equation - CR corrosion rate - by Pomeroy (1977) is given by

$$CR = 11.4 * k * \phi_{sw} * (1/a) \quad (1)$$

where

11.4 = empirical factor

C = average rate of corrosion of concrete by acid (mm per year)

k = coefficient for acid reaction, accounting for estimated fraction of acid remaining on wall

0.8 for S (increase of sulfide concentration) ≤ 1.0

0.7 for $1.0 \leq S \leq 5.0$

0.6 for $S > 5.0$

ϕ_{sw} = flux of hydrogen sulfide to pipe wall ($\text{g/m}^2 \cdot \text{h}$)

a = coefficient for alkalinity of concrete, normally 0.16.

The CR corrosion rate can be simplified and formulated as:

$$CR = 0.4 * S_1 \quad \text{for part-flow pipes with capacity} < 10 \% \quad (2)$$

$$CR = 1.0 * S_1 \quad \text{for part-flow pipes with capacity } 10 \dots 50 \%. \quad (3)$$

Corrosion is a function of exposed perimeter, surface width of stream, velocity, and dissolved sulfide. From the corrosion rate the so-called critical sewer length L_c can be calculated as follows, from Pomeroy (1977)

$$L_c = 3.6 * v * t_c \quad (4)$$

where

L_c = critical sewer length (m)

3.6 = empirical factor

v = velocity of waste water (m/s)

t_c = flow period (s).

The rate of corrosion of concrete pipes has been expressed by Thistlethwayte (1972) using the theoretical equation

$$CR = 19.9 * 10^6 \frac{K_{sa} * p_{H_2S} * A_{sa}}{z * \rho * A_{aw}} \quad (5)$$

where

K_{sa} = rate of absorption of H_2S on pipe wall ($\text{kg/m}^2 \cdot \text{h}$)

p_{H_2S} = partial pressure of H_2S (ppm)

A_{sa} = ratio of surface width of waste stream to exposed perimeter of pipe wall above water surface (m^2/m)

z = of cement content of concrete (kg/m^3)

ρ = density of concrete (kg/m^3)

A_{aw} = exposed perimeter of pipe wall above water surface (m^2/m).

Using the equations 1 and 4, the hydraulic and sulfide data of a number of examples were used to calculate the predicted corrosion and the critical sewer length in Table 1.

Table 1: Calculation of predicted corrosion rates for different sewer conditions

Parameter	Sewer Condition			
	I	II	III	IV
Rate of flow (l/s)	50	1.700	1.700	50
Part-flow capacity (%)	≤ 10	10...50	50...80	80...100
Velocity of waste water (m/s)	0.50	1.40	1.40	1.0
Rate of corrosion (mm/year)	0.72	4.0	18	4.0
Critical sewer length (km)	1.5	20.4	20.4	2.4

The calculation based on the values established by Pomeroy (1977) allow an exact prediction to be made for the expected level of biogenous sulfuric acid attack in sewers. The examples clearly shows that concrete pipes do not only need to be protected against acid corrosion when exposed to extreme conditions.

3 Time-accelerated laboratory tests

To simulate resistance to biogenous sulfuric acid attack, a very simple test procedure has been developed that provides an authentic demonstration of how sulfuric acid attacks in dirty and combined waste water systems. In these tests waterlogged specimens are weighed and stored in plastic containers. Each container is filled with diluted sulfuric acid (pH value 1, 2 or 4.5). The pH value of the acid bath is kept constant over 35 days. This test procedure is suitable for a decision on service life under various working conditions. Figure 4 illustrates the test equipment for the acid storage of concrete specimens.

After 6, 13, 20, 27 and 34 days the specimens are removed from the solution, then stored under running water, vigorously brushed off and all loose parts removed. After this the specimens are weighed and stored, following the same procedure. This operation corresponds to practical conditions of cyclic flooding in combined waste water sewers after downpours. The acid is circulated permanently. In order to keep the pH value constant, concentrated acid is added. The acid bath is restored after each storage cycle.

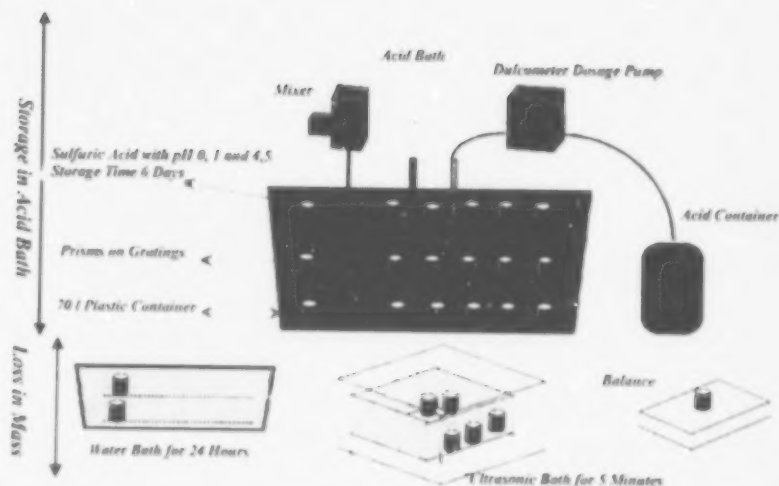


Fig. 4: Test equipment for time-accelerated testing of the corrosion by biogenous sulfuric acid attack

4 Correlation between predicted and observed corrosion

It is a well-known fact that corrosion in the crown of pipes may be up to 80 % greater than the average in exposed pipe wall areas. Use of the Pomeroy equations for the prediction of generating corrosion proves a helpful tool in the design of sewers. The predicted corrosion using Pomeroy's equations and carrying out time-accelerated tests in diluted sulfuric acid is compared with the observed corrosion in real concrete sewers. This is illustrated in Table 2.

Table 2: Comparison of predicted corrosion with observed corrosion

Parameter	Type of corrosion			
	I	II	III	IV
Residual weight (%)	-	94	84	95
Undamaged cross-section (%)	86	85	74	88
Residual compressive strength (%)	86	84	72	81
Acid penetration depth (%)	14	12	19	10

I - Calculation of predicted corrosion using Pomeroy's equation

II - Corrosion after 35 days storage in diluted sulfuric acid pH 2

III - Corrosion after 35 days storage in diluted sulfuric acid pH 1

IV - Observed corrosion in sewer

For laboratory tests, different concrete samples taken from the whole perimeter

of the wall were removed by coring and then measured. After removal of the soft surface layer, some of them were stored in diluted sulfuric acid to determine the corrosion in time-accelerated laboratory tests. All specimens were tested for residual weight, undamaged cross-section, residual compressive strength and acid penetration depth.

The sewer under investigation is located in the central part of Germany. The sewer conveys both domestic and industrial waste water. The trunk is 8.8 km long. Of this, 2.8 km is unreinforced concrete pipe, 1,200 mm in diameter. The original wall thickness was 120 mm. The trunk was put into operation in 1976. Visible corrosion occurs in the upstream sections of pipe, especially at the springline. However, the measurable corrosion at the crown is usually greater than the average corrosion of the pipe.

The trunk was designed and constructed with a compressive concrete strength of 35 N/mm², and made of ordinary Portland cement type CEM I 32.5R. The cement content was 340 kg/m³ and the water-cement ratio 0.40. In order to reduce the cement content, fly ash was added.

The total flow period of waste water through the trunk amounts to 10.5 hours. The part-flow capacity alternates between 10% and 30%. After downpours the sewer is full-flow. The concentration of hydrogen sulfide in the atmosphere was measured at 1.6...2.8 ppm. The pH value of the thin wet film on the pipe wall surface averaged 3.5.

5 Conclusions

Biogenous sulfuric acid corrosion is one type of corrosion in sewerage systems. Chemical attack on the pipe wall surface takes place above the effluent line. The corrosion in the crown of pipes may be greater than the average for exposed areas. Values of more than 1.0 mg/l for total sulfide content in waste water and of more than 0.5 ppm for hydrogen sulfide concentration in the sewer atmosphere point to the possibility of high-level chemical attack. The generation of biogenous sulfuric acid depends on pipe diameter, sewer length, pipe gradient, pipe ventilation, turbulence and other aspects of construction.

Due to the fact that sewers worldwide are increasing in size and that flow distances and periods are getting longer, the necessity of monitoring biogenous sulfuric acid attack is becoming more urgent. Sewers are large-scale investments with a planned service life of at least 80 years. Special consideration must be given to problems of sulfuric acid corrosion.

Investigations by Parker (1951), Pomeroy (1977), Thwistlethwayte (1972), Bielecki (1987) and others have led to the establishment of equations for predicting corrosion of sewers and critical sewer length. Although the general rate of corrosion can vary considerably, it is possible to predict corrosion by using both theoretical equations and time-accelerated laboratory tests. For an exact prediction of service life, the main parameters of waste water quality, pipe atmosphere and concrete specifications must be available.

The observed corrosion of a 22-years-old unreinforced concrete pipe was simi-

lar to the average predicted corrosion using Pomeroy's equation. The laboratory simulation tests provide a useful tool for determining the service life of mortar and concrete in sewer atmospheres. The designers can provide a protective concrete cover over the reinforcement steel.

The very simple time-accelerated laboratory tests of storage in diluted sulfuric acid give results which can easily reproduced. Extensive laboratory tests showed that storage in diluted sulfuric acid for 35 days with 5 cycles corresponds to real-time stresses in ordinary combined sewers of about 20 years.

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5 INVESTIGATION OF CONCRETE JETTIES AT CFB ESQUIMALT AND PREDICTION OF FUTURE SERVICE LIFE

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Abstract

The concrete jetties at the Canadian Forces facilities at Esquimalt and Nanoose Bay are of similar design and age. They consist of a 300 mm reinforced concrete topping which acts compositely with square box girder beams which span between pile bents. The concrete topping at both jetties has undergone severe cracking, which has been attributed to drying and autogenous shrinkage aggravated by the use of epoxy coated reinforcement. Many of the cracks are aligned with the reinforcement and have created reservoirs for chloride ions concentrated very near the reinforcement. Condition surveys indicate that while the chloride ion concentrations adjacent to the reinforcement are high, no corrosion damage has occurred to date. Various analytical models have been investigated to estimate the future service life of the structures and the potential effectiveness of the remedial work such as the application of surface sealants.

Keywords: Epoxy coated reinforcement, cracking, predicted service life, sealants.

1 Introduction

C-Jetty, which is located at CFB Esquimalt and Ranch Point Jetty at Nanoose Bay in British Columbia were investigated to evaluate the condition of the concrete and the state of the reinforcement in the reinforced concrete decks.

These investigations were completed in 1996 at C-Jetty and in 1993 at Ranch Point Jetty. They were performed in part to obtain baseline information on the Jetty conditions and also to evaluate the nature and severity of cracking which had been

observed in the concrete deck surface, with a view to the design of appropriate remedial procedures.

2 Structure

Both Ranch Point and C-Jetty were designed in a similar manner. The jetties consist of rectangular box girders supported on pile bents at 19 m centres, overlain by a 200 to 250 mm thick reinforced concrete topping. Both jetties were constructed in approximately 1989-1990. Some known differences in the jetties are that epoxy coated reinforcement was used in both the top and bottom mats in the topping concrete at Ranch Point Jetty, while at C-Jetty the top mat was epoxy coated and the bottom mat was plain reinforcing steel. Secondly, the Ranch Point Jetty deck is on cathodically protected steel piles while the C-Jetty deck is on concrete piles. These two details have had considerable effect on the copper-copper sulphate half cell data measured on each deck.

3 Observations

Both jetties were observed to be in a very similar condition. In general, there was little evidence of deterioration of either the concrete, or the reinforcing steel in the decks. However, the topping concrete on each deck contained severe shrinkage cracking. Typical examples of the cracking are illustrated in Figures 1 and 2.

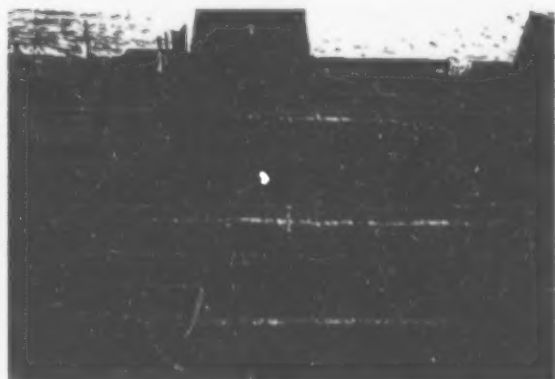


Fig. 1: Portion of C-Jetty showing cracking

3.1 Deck cracking

The deck cracking in C-Jetty was mapped for most of the jetty surface and the floor of the Ship Repair Unit (SRU) Building. The cracking is extensive over the deck surface and in many areas was found to be aligned along the axes of the deck; i.e., the cracks are predominantly longitudinal and transverse to the length of the fingers. This is illustrated in the photograph shown in Figure 1 which were taken at approximately 8.00 am as the morning dew dried from the deck surface. The cracks were accentuated by the presence of moisture.

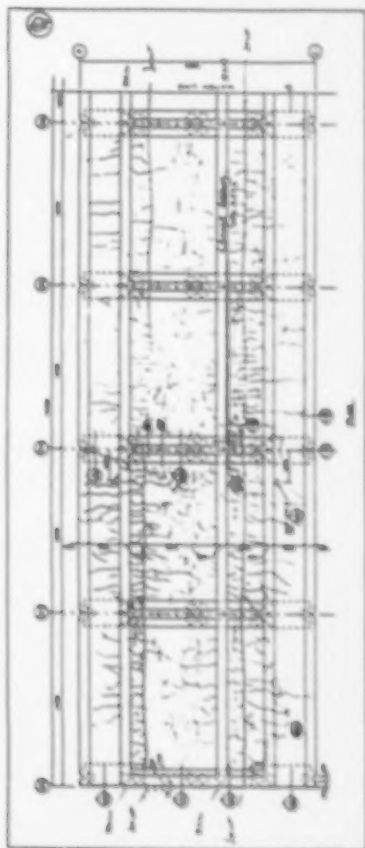


Fig. 2: Cracking on West finger of C-Jetty

The cracking in C-Jetty and the SRU building is very similar to the cracking observed at the Ranch Point Jetty at Nanoose. For both structures, it appears that the cracking is primarily the drying shrinkage and the plastic shrinkage induced, aggravated by the use of epoxy coated reinforcing steel. Previous work by Litvan (1989) has indicated that there is reduced bond of concrete to epoxy coated bars and that this may contribute significantly to the extent and width of cracking. This is reinforced by the cores undertaken which revealed that many cracks appear to originate at the location of the epoxy coated reinforcing steel.

3.2 Concrete cores

Concrete cores were extracted from eight locations on C-Jetty. At least three cores were extracted at each location. These cores were used for a variety of laboratory tests including compressive strength, depth of carbonation, chloride ion profile and density.

3.2.1 Compressive strength and density

The average strength for each location is listed in Table 1. It is understood that the specified compressive strength was 35 MPa at 28 days. As shown in Table 1, the cores from all areas tested (except at location 7) meet this criteria. At location 7 the compressive strength is very close to the specified 35 MPa value.

Table 1: Compressive strength and density

Location	Density (kg/m ³)	Compressive Strength(MPa)
1	2367	47.2
2	2332	46.3
3	2408	62.0
4	2400	57.9
5	2322	35.7
6	2401	48.3
7	2340	34.6
8	2397	35.7
		AVG=46.0

3.2.2 Depth of carbonation

Treating the core samples with phenolphthalein indicator solution showed that no measurable carbonation has taken place to date. It is, however, recommended that the depth of carbonation be determined at regular intervals, particularly at locations where cracks intercept the reinforcing steel. If the concrete around the reinforcing steel becomes carbonated, then the concrete's ability to inhibit corrosion may become compromised.

3.2.3 Chloride ion content

The samples for chloride ion determination were obtained by dry drilling the cores at the desired depths perpendicular to the core axis. Typically samples were obtained at depths of 25 mm, 50 mm, 75 mm and 150 mm. The procedure used for determination of the chloride ion content was the "water soluble" method. The chloride ion concentrations have been calculated for a concrete with an average density of 2350 kg/m³ and cement content of 350 kg/m³. The results are presented in Table 2.

Additional samples were also obtained by dry drilling to specified depths into the in-place topping concrete. Companion samples were obtained in this manner both at the location of the cracks and immediately beside cracks (Samples 9 to 11).

According to the American Concrete Institute, ACI 318-95, recommended limit for water-soluble chloride ion content in conventionally reinforced concrete in a moist environment and exposed to chlorides is 0.15 % by mass of cement. At levels higher than 0.15 % there is a significant increase in the potential for chloride induced corrosion of the reinforcing steel.

As shown in Table 2., there are many areas where the chloride contents are below the detection limit of 0.007 %. However, near the surface of the concrete and in the cracks, the levels can be much higher than the ACI 318-95 recommended limit.

1. In the sound, uncracked concrete the chloride concentrations were measured to vary from 0 to 0.215 % at a depth of 25 mm. At depths of 50, 75 and 150 mm the chloride levels were in many cases either undetectable or relatively low.
2. At the cracks in the concrete topping, however, the higher chloride levels can occur at the base of the crack as shown for Samples 9 and 11. Because many of the cracks have originated at the reinforcing steel, elevated chloride levels have been found at the steel. As shown in Samples 9 and 11, these levels are 3 to 5 times the limit for controlling corrosion recommended in ACI 318.

3.3 Half cell potential measurements

Copper - Copper sulphate half cell measurements were obtained for the four areas shown in the C-Jetty plan. The half cell data is typically in a range which is less than -350 mV. The exceptions are the area East of the SRU Building and along the rail on the two fingers. The elevated potentials along the rails can most likely be explained by the corrosion occurring at the rails and base plates within the rail trench and the considerable surface area of steel available for corrosion sites. The rail trench is filled with hot mix asphalt concrete and therefore does not benefit from the usual corrosion inhibiting qualities of alkaline concrete. In addition, the trench can act as a collector for the surface runoff and chlorides from both salt spray and deicing salts.

Test results for the area to the east of the SRU Building which also exhibited high half-cell potential readings (ie; -600 to -800 mV) are not as easy to explain. They do not, however, appear to be the result of corrosion of the reinforcing steel. Because of the consistency of the signal, it appears that a stray current within the reinforcement has been detected. Inspection of the area below the slab revealed that a ground wire is attached to many of the piles and this may be contributing to a portion

of the measured signal. This area will be investigated further since (as discussed by Clifton (1991)) stray currents, may accelerate the corrosion process.

Table 2: Water soluble chloride ion content

Sample	Depth (mm)	Chloride Ion Content ⁽¹⁾ (% by mass of cement)	Comments
1B	25	0.215	-samples from core
	50	0.034	
	75	<0.007	
	150	0.045	
7A	25	0.164	-samples from core
	50	<0.007	
	75	<0.007	
	150	<0.007	
9	0-25	0.404	-drilled samples on crack
	25-50	0.444	-refusal (steel?) at 53 mm
	50-53	0.710	-outside SRU door
9A	0-25	0.125	-drilled beside crack and sample 9
	25-50	0.013	
	50-75	<0.007	
	75-125	<0.007	
9B	45-53	0.252	-drilled beside crack
			-refusal (steel?) at 53 mm
10	0-25	0.356	-drilled on crack
	25-50	0.172	
	50-75	0.018	
	75-97	<0.007	
10A	0-25	0.167	-drilled beside crack near sample 10.
	25-50	<0.007	
	50-75	0.014	
	75-100	<0.007	
11	0-25	0.451	-drilled on crack
	25-50	0.350	
	50-60	0.526	
11A	0-25	0.543	-drilled beside crack
	25-50	0.095	
	50-75	0.014	
	75-100	<0.007	
	100-112	<0.007	

- (1) Chloride Content is expressed as water soluble % by mass of cement for an estimated concrete density of 2350 kg/m³ and cement content of 350 kg/m³.

4 Predicting the future service life

As discussed above, the C-Jetty is showing no signs of distress due to corrosion of the reinforcement at the present time. However our opinion is, that the service life of the deck may be compromised by the extensive cracking which has developed in the topping concrete, and subsequent chloride contamination at crack locations. This increases the possibility of chloride induced corrosion occurring and shortening the service life of the structure.

Various means of remediation were reviewed including the possibility of chloride removal either by flushing the cracks with clean water or by electrochemical means. Neither was determined to be feasible. The flushing did not change the chloride concentrations significantly and electrochemical removal of the chlorides is not considered appropriate because the reinforcement is epoxy coated, and the developments of an electropotential driving force for chloride removal would be weak. Consequently, it was elected to attempt to apply a low viscosity sealer capable of penetrating the cracks. It was hoped that the sealer would reduce the permeability of the surface and provide some water repellantcy in the cracks, thereby reducing the quantity of moisture and salts reaching the reinforcement.

To assist in the process of deciding whether or not to apply a sealer we have used the analytical model developed by Yokozeki *et al.* (1997). Several models including those developed by Weyers *et al.* (1994) and Maage *et al.* (1997) have been reviewed. The model by Yokozeki *et al.* (1997) appears to best fit the example of the C-Jetty.

Many of the Service life models are based on the progress of chloride-induced corrosion and deterioration. This is illustrated in Figure 3. The model is separated into two parts.

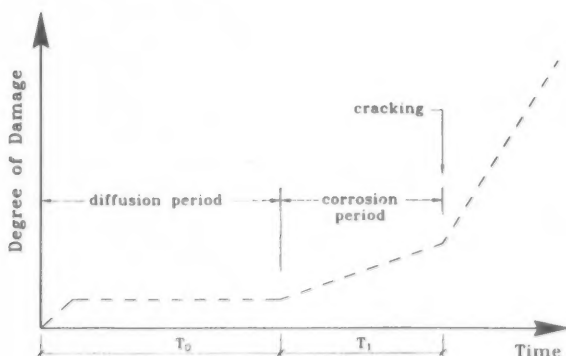


Fig. 3: Graph illustrating corrosion damage vs time

T_0 = the time for onset of chloride induced corrosion; and
 T_1 = the time for corrosion and crack development.

The time for the onset of chloride induced corrosion is based on Ficks Law of diffusion and is given by:

$$T_o = \frac{1}{D_{cl}} \left[\frac{L}{2 \operatorname{erf}^{-1} \left\{ 1 - \frac{C_{cr} - C_i}{C_{ocl} - C_i} \right\}} \right]^2$$

- D_{cl} = Diffusion coefficient of chloride in concrete. (mm/year)
 C_{cr} = The limit amount of chloride ions against corrosion
 C_i = Initial chloride ion content in concrete
 C_{ocl} = Chloride ion content in concrete surface
 L = Concrete cover thickness (mm)

For the C-Jetty, we have estimated the diffusion coefficient, D_{cl} from *Maage et al.* (1997) to be 30 mm/year

- C_{cr} = 0.15 % by mass of cement
 C_i = 0
 C_{ocl} = 0.4 % by mass of cement
 L = 50 mm

With this data the predicted time to the onset of corrosion is 42 years. However, if we assume that the cover has been significantly reduced as a result of the cracking the time T_o is reduced as follows.

Cover L (mm)	0	10	20	30	40	50
T_o (years)	0	1.7	6.8	15.3	27.2	42

Secondly, the time for initiation of crack development is determined from the following equation

$$T_1 = \frac{\text{Critical Corrosion Amount (mg/cm}^2\text{)} W_{cr}}{\text{Rate of Corrosion (mg/cm}^2\text{/year), } W}$$

where $W=742/L$ (from Table 1 of Yokozeki *et al.* (1997))

and $W_{cr} = 1.841\phi(\phi - 8.661) + 145.1\alpha^{-1.194} + 3809D^{-0.8361} + 10.60x_1 - 72.30$

where ϕ = creep coefficient = 0.4

α = vol. expansion ratio for corrosion products = 3.2

$D = 360^\circ$

$X_1 = L/d$ (in this case)

As for T_0 , we have computed T_1 at various concrete cover thickness.

Cover L (mm)	10	20	30	40	50
T_1 (years)	.09	0.4	1.0	2.0	3.1

Thus if we assume that concrete cover is 50 mm then the predicted service life will be;

$$T = T_0 + T_1 = 42 + 3.1 = 45.1 \text{ years}$$

However, if as we suspect the concrete cover has been reduced because of the extensive cracking in the surface then the service life is substantially reduced. For example if we say that the actual cover is more typically 20 mm, then;

$$T = 6.8 + 0.4 = 7.2 \text{ years}$$

It is difficult to accurately define the reduction in effective concrete cover produced by cracking. However, the calculations of predicted service life show that even a small reduction in cover thickness results in a significant reduction in the service life prediction. It is likely that damage, when it occurs, will start at the larger cracks which permit greater access of chlorides moisture and oxygen to the reinforcing steel. Thus corrosion will likely occur initially at a few localized areas. At these locations, the greatest resistance to corrosion-induced damage is likely the epoxy coating. Thus the epoxy coating life expectancy could be substituted for T_0 in the preceding calculation. In studies by others (memo 1992; Hearn 1992; Weyers 1997) it has been indicated that typical epoxy coating life expectancy is in the range of 5-15 years, five years in warm salt water environments and 15 years in colder environments. Victoria has a relatively temperate climate but the Jetty receives substantial applications of deicing salt during the winter. Therefore, an epoxy coating life expectancy of the order of 8-10 years is anticipated in this environment.

According to the study by Weyers *et al.* (1997), coating debondment occurs mainly as a result of attack by the moisture available in the concrete. Therefore, the two activities of chloride diffusion and coating debondment likely run concurrently. Weyers also showed that 20 years following the successful sealing of several bridge decks, the chloride contents were reduced to approximately 50 % of the original concentration. This reinforces the recommendation to seal the C-Jetty.

5 Conclusions

1. The deck concrete at the C Jetty has been severely affected by shrinkage cracking. The cracking appears to have been increased by the use of epoxy coated reinforcement.
2. The cracks are heavily contaminated with chlorides and act as conduits to the reinforcement.
3. It is postulated that the cracking has reduced the effective concrete cover on the deck.

4. The analytical model developed by Yokozeki *et al.* (1997) indicates that there may be a significant reduction in the service life; i.e., a reduction in the time to corrosion caused cracking, and the need for repairs.
5. It is recommended that the deck be treated with a sealer capable of penetrating the cracks. It is expected that this will seal the cracks and help to restore the effectiveness of the concrete cover.

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6 THE REAL SERVICE LIFE OF ROAD BRIDGES IN SWEDEN - A CASE STUDY

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Abstract

To clarify the real condition of a country's bridge stock and their structural members at a given time, reliable regular and systematic inspections are needed. Accurately documented, these inspections are essential for the road administration authority in the planning of the necessary maintenance and repair works on the existing bridge stock, thus contributing to an overall cost-effective bridge management.

The Swedish Road Administration has, at the present, a relatively intact and well-organised bridge archive system. Most of the former bridge inspections performed on the public road network after the nationalisation of the Swedish roads in 1944, are documented and stored in different archives throughout the country. The information from these inspections has been fed into a Microsoft Access database called BEA (Bridge Element Analysis), with the purpose to explore the possibility to determine the damage occurrence, damage growth and the real service life for bridge structural members.

In this case study, the database is used to examine, if certain information and factors like geographical location, traffic, and weather data at the time of casting, affect the service life of bridge structural members thus the service life of bridges. Based on the information stored in the database, the service life of bridge structural members in certain service life conditions may be estimated. For example, the real service life of bridge members as a function of the outside air temperature at the time of the concrete casting. The information stored in the database may also be used in the development of a damage growth model based on the assessed condition at the time of inspection.

Keywords: Bridges, service life, bridge structural member, bridge management, database, estimated service life, damage type, damage cause.

1 Introduction

1.1 General

This paper is the result of a case study on the real service life of some 270 bridges located on the Swedish National road-network. However, the results from the extended study during 1998, comprising a total of 370 bridges are not presented in this paper.

Bridges are an important part of a nation's road system. Maintaining and repairing existing bridges is a major economic concern for many governments and local authorities around the world. Many bridges are built with outdated technical codes and demands. The service life of a bridge is often defined as the time from construction to the time a critical limit state or property has been reached. It is generally expected that during their service life, bridges can fulfil certain demands such as traffic safety, continuous traffic flow and a designed load bearing capacity. Regular and systematic inspection of the existing bridge stock should be performed in order to verify that such demands are met at all times.

With the 1944 nationalisation of the public roads in Sweden, information about most of the regular bridge inspections have been carefully documented and filed by the Swedish National Road Administration in different archives. Damages are rated in condition grades on a scale between zero and three.

1.2 Background

The research program "The durability, management, repair and life cycle cost of concrete structures" conducted at the Royal Institute of Technology was initiated by the Swedish National Road Administration in 1991. As the program title suggests the program consists of a number of research projects. The project "Actual durability and service life of Swedish concrete bridges" was started in late 1992. The aim of the project was, initially, to establish the research possibilities of the vast amount of information in the archives and data in the bridge management system of the Swedish National Road Administration. The project resulted in the later part of 1997 in a Licentiate Thesis, "The Real Service Life of Bridges".

1.3 Aims and scope

The second stage of the project was a further investigation in the use of the bridge information available with the aim of developing deterioration models. This has been achieved for bridge structural members in certain service conditions. In the future, bridge designers and inspectors would predict the performance of certain bridge structural members in certain service environments according to type and cause of the damage.

Service life models in combination with a relational database containing actual durability information would result into a tool for assessing and predicting the actual service life. If such a model could be achieved, it would give researchers and owners a precious tool in the planning of maintenance works, design of the different structural parts and component and feedback for future codes and regulations.

The development in time of the condition grade can be a way of predicting the service life of bridge components, contributing to cost effective bridge management and improved bridge component design in the future.

2 Method

Sweden has many different local climate conditions. In the north of Sweden there are few yearly frost-cycles, mainly because the temperature is always below zero degrees Celsius throughout the winter season while it can oscillate around zero degrees in central and southern Sweden leading to several frost-cycles. This implies better frost-resistance performance of the concrete in the central and southern parts of the country.

De-icing salts are used on many roads during the cold season of the year. Because of the various climate conditions in Sweden, bridges in the central and southern part of Sweden are especially exposed to de-icing salts. It became evident in the early part of the study that to minimise the service life variables, bridges should be studied on geographically close road sections. 15 different road and highways sections with 188 bridges were selected in the central part of Sweden. In order to be able to study how different climates affect the bridge structures, the geographical extent should be increased with road sections in different parts of the country. This resulted in the gathering of information on 82 bridges in the northern part of Sweden. The roads in that particular county are de-iced using very little salt or no salt at all which is an interesting factor to study.

2.1 The information gathering process

2.1.1 Extent

In order to gather the information in an efficient way a strategy to find relevant information was prepared. During the first part of 1993, detailed checklists and routines for the gathering process were produced. This was essential in order to minimise the risk for errors and loss of information. The gathered information was placed in project files for each road section in the investigation. This material is unique, and should be an important asset for many research projects.

The limitation of the road sections, in both length and the amount of bridges were adjusted so that traffic flow and climate have similar conditions. This was made to make road sections comparable and thus enabling the study of certain factors imposed by the surrounding environment that might affect the service life of structural members. In 1997, the investigation included 19 different road sections with a total of 270 bridges. The results and conclusions were presented in the Licentiate thesis "Konstbyggnaders reella livslängd", the Royal Institute of Technology, Stockholm.

The R&D project was continued and during 1998, 100 bridges on four road sections in the western and eastern parts of Sweden were added to the investigation, thus covering two new different parts of Sweden.

2.1.2 Selection of information

The gathered information for each bridge can be divided into three different groups:

- General information on the bridge
- Inspections and repair works performed on the bridge
- Information from the construction stage and the final inspection

All performed inspections and repairs by the Swedish National Road Administration were summarised in the inspection- and maintenance sheet of the bridge and archived. Even these have been copied, when available, for the bridges included in the investigation. Information given is notes the inspectors have taken during inspection or the maintenance work and the date they were performed.

The basis of the inspection- and maintenance sheet is "the full inspection report". That report gives information, in addition to that in the summarised inspection- and maintenance report, on for example suggestions of bridge maintenance measures or activities and who should carry them out.

The full inspection reports were not always easy to find. Unfortunately, in many cases only the inspection- and maintenance report had been saved.

The documentation from the performed special inspections is often in the form of tables and diagrams and have been found both at the central and regional archives of the Swedish National Road Administration. Values from the chloride profiles have been inputted the database BEA for analysis. Other gathered information, on bridges in the investigation found in the different archives, was copied and placed in the project files. Concrete castings are documented in concrete casting reports. The report includes facts about mixing conditions, quantity, concrete quality and meteorological conditions at the time of concreting.

At many concrete castings, concrete tests are taken for analysis. The concrete samples are related to a testing record in the concrete casting reports. Some tested qualities are compressive strength, water tightness and frost resistance of the concrete.

Examples

A lot of the gathered information is of historical value. One can, for instance, look back in time and see how certain parameters have developed in reality. An example of this is the development of the real (test results) compressive strength of the concrete in certain structural members.

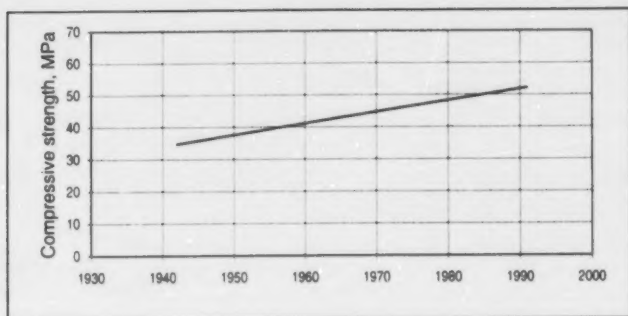


Fig. 2a: The real development of compressive strength (test values) according to documented concrete castings for deck slabs in the investigation

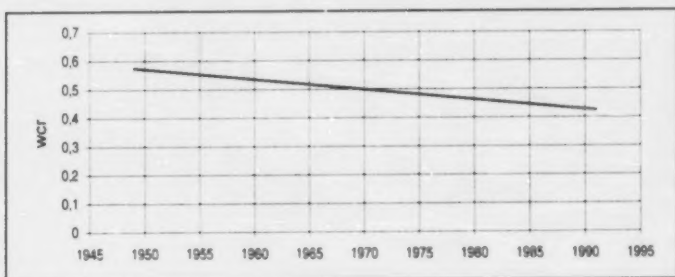


Fig. 2b: The average development of water cement ratio, wcr, throughout the years according to all documented concrete castings in the investigation

Another example is how the water-cement ratio, (WCR), has decreased throughout the years. All gathered information on the WCR are originate from the concrete casting journals from the investigated bridges that were found in the different bridge archives of the Swedish National Road Administration

2.1.3 Other sources of information

One of the aims of the R&D project was to identify possible sources of information within the Swedish National Road Administration. A number of available databases were used extensively and successfully in the information gathering process. These are:

- Road Data Bank (Vägdatabanken VDB)
- Bridge Data (Brodata)
- Central and Regional Bridge Archives
- Road Museum in Borlänge

Another important source of information was gathered during interviews with experienced personnel working with the Swedish National Road Administration. Bridge inspectors, bridge engineers, archivists and historians helped to complete the missing ends, which occurred during the entire time.

3 Bridge element analysis, BEA — Relational database

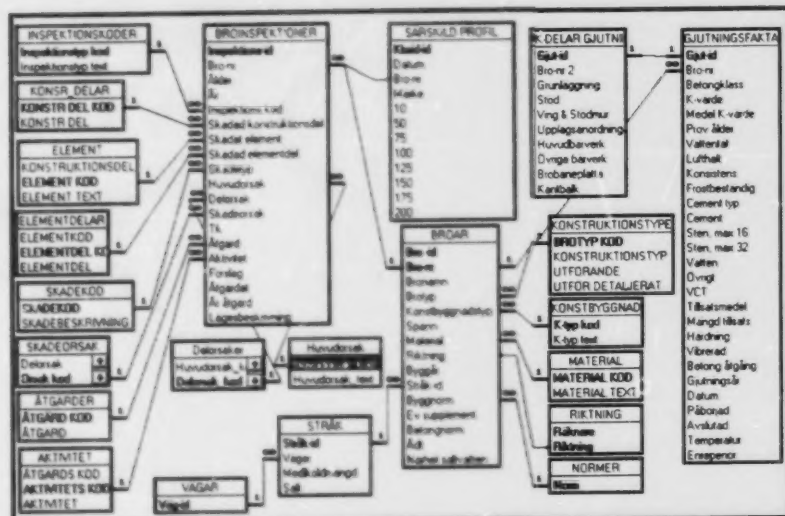
It became clear at an early stage that Brodata, which is the bridge database of the Swedish National Road Administration, could not be used with the aims and goals of this R&D project. Brodata does not contain any information on performed bridge inspections prior to 1987. A new database should be created. As all available and relevant concrete information such as wcr, designed and real compressive strength, entrained air, weather data from the time of casting, maintenance and repair work performed on the structures and more was gathered, the choice fell on using Microsoft ACCESS relational database.

The Windows - ACCESS database, BEA (Bridge Element Analysis), specially developed for the needs and requirements imposed by this research project, deliberately uses the same codes, definitions and terms as used by the Swedish National Road Administration. The database is described in detail under chapter four in the Licentiate Thesis "Konstbyggnaders reella livslängd".

The process of creation and development of the Bridge Element Analysis, BEA, database was achieved in five major steps:

1. Step one: Planning; What will the results be used for? How to design the tables and relations for an optimal database structure? Why certain data will be selected for the analysis?
2. Step two: Creating tables and relationships; All tables and relations were created in such a way that data could be combined after needs.
3. Step three: Creating queries; Queries can cross-examine data in one or more tables, after the needs of the project.
4. Step four: Creating forms; Data input needs a number of forms, which make the database user friendly.
5. Step five: Creating reports; When all data has been put into tables, queries are completed with reports. New reports have been created continuously, according to increased search parameters.

The database BEA has great development possibilities. New objects can be created without difficulties. The performance can be improved if that becomes a priority, for example, through division of the database or better query design. Many queries have developed during the investigation. This is mainly due to the many questions of issues and reports that have occurred throughout the project.



4 Damage picture in the investigation

2898 performed bridge inspections on the 270 investigated bridges have brought forward 2165 damage remarks where type- and cause of damage is stated. Even if the result is not significant for the entire country, they are giving a clear indication of the general condition of bridges in the central and northern parts of Sweden.

The first step is to present how the 2165 damage remarks are divided on the different structural members of the bridges. Later the some types of damage over-represented on the structural member level for all condition classes will be presented. The last step is to present, which causes of damage, for all condition classes, that lay behind the most common damage types.

4.1 The damage picture on structural member level

As previously stated, the bridge inspector must record certain damage information during the inspection. The extent of the data depends on the type of performed inspection. The requirements are established in the bridge inspection manual of the Swedish National Road Administration.

4.1.1 Assessment of condition classes (CC)

Two important requirements in the damage documentation process are the measurement and condition assessment of damages. This is done for damaged structural elements in the following two stages:

- Stating the physical condition in terms of measurement and measured value
- Assessment of the functional condition in terms of condition classes

The physical condition is determined with reference to the development of previous or new damages and certain known deteriorating processes. The different methods of measurement that are to be used for a particular type of damage are described in publication 1993:35 (in Swedish) or publication 1996:038(E) (in English). The physical condition of a damaged structural element can then be described using the variable defined for each method of measurement.

The functional condition of a structural member is described by the bridge inspector in terms of condition classes. The condition class describes to what extent a certain structural member satisfies the designed functional properties and requirements at the time of inspection. It can be said that the assessment of condition classes is composed on previous and current measured values (the physical condition) and the inspector's competence in the propagation of different deterioration processes. The condition class (CC) for a structural member can be registered on a scale of four. The scale implies that, at the time of inspection, the structural member was considered to have:

Table 4: Assessment of condition classes for bridge structural members

Condition class (CC)	Assessment
3	Defective function
2	Defective function within 3 years
1	Defective function within 10 years
0	Defective function beyond 10 years (No damage at time of inspection)

4.2 The damage picture in the investigation on structural member level, all condition classes

With this search criteria, all damage remarks will be expressed as a percentage, even those damages that have been given the condition class "0".

Interesting in the context is that the structural members "edge beam", "support" and "deck slab" together answer for 47% of all remarks, regardless of year of construction, type- and cause of damage.

The result confirms the hypotheses that these three structural members can become the largest maintenance problem for the Swedish National Road Administration in the future. The service life problem on Swedish bridges is directly dependent upon the physical condition development for these three structural members.

The structural member's "slope and embankment end" and "parapet" together answer for 26% of all remarks.

The restoration of embankment ends is a frequent maintenance problem for the Swedish bridge management. Steep slopes and embankment ends, above all on the older bridge stock, have such inclination that it seems like the former bridge design-

ers wished to defy the law of gravity.

On most Swedish bridges the parapet is fixed to the edge beam. This is, in many cases, an unfortunate solution. The structural member "parapet" is more frequently involved in collisions, and this can lead to a domino effect where the edge beam is damaged.

Condition class 3 means that, at the time of inspection, the structural member has entirely lost its function. The structural member "parapet" is over-represented in the group of condition class 3. It answers for 22% of all reported damages; most of them caused by collisions. The structural member "edge beam", "support" and "deck slab" together answers for 27% of all damages in this particular group. Worth noting is that the structural member "primary load-bearing element" has managed fairly well in the investigation with only 1% of all reported damages with assessed condition class 3.

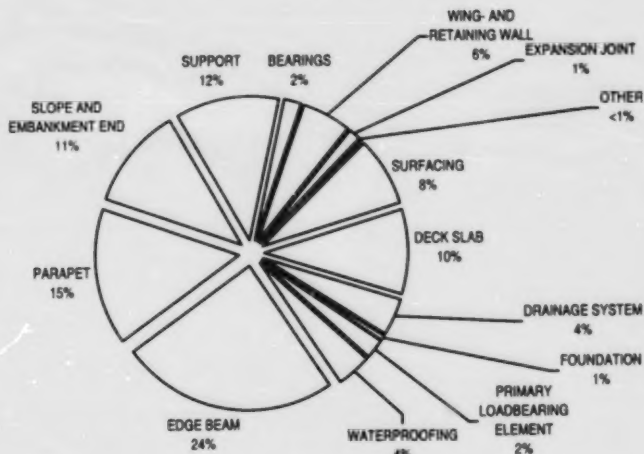


Fig. 4: 2165 damage remarks, all condition classes, and percentage division by the bridges' structural members

5 Damage types and causes

Bridges consist of a number of structural members, which, in their turn can consist of a number of structural elements. Even structural elements can consist of a number of element parts. When a damage is detected, the damage is documented with information about the type of damage and the cause of the damage.

5.1 Types of damage

The damage types used in the investigation follows that used by the Swedish Bridge Management System, SAFEBRO. Initially, the investigation of 270 bridges, 2165 reported damages were registered. "Corrosion" as a damage type can even be related to other steel components such as railings.

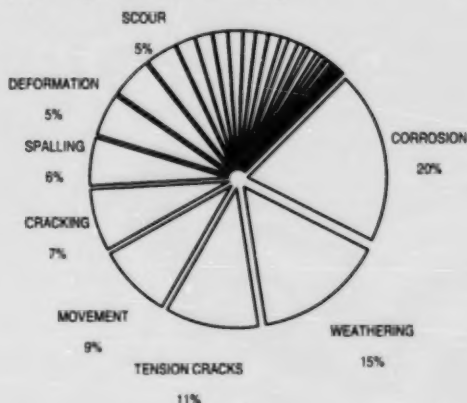


Fig. 5a: Distribution of the 2165 (>5%) damage reports in the investigation after type of damage

5.1.1 Causes of damage

If an inspection report contains damage remarks, the bridge inspector must establish the probable cause of the reported damage. This will be accomplished in three steps. First, the primary cause has to be determined. These can be one of the following:

- Defective construction
- Service condition
- Environmental action
- Accident
- Defective design
- Defective maintenance

Each of the primary damage cause is subdivided in their turn into secondary and tertiary causes.

Over 40 percent of all the damages in the investigation are related to "Environmental action" and remarkably only one percent to "Defective design".

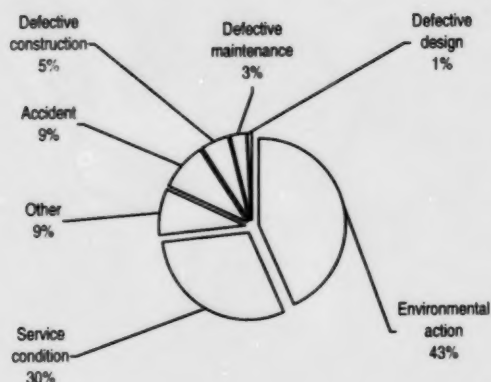


Fig. 5b: Distribution of the primary causes of damage in the investigation

Table 5: Environmental action as primary cause of damage. The distribution of the secondary and tertiary causes of damage in the investigation

Environmental action (937)			
Environmental Action (725)	Physical attack (163)	Chemical attack (30)	Biological attack (19)
Environmental Action (183)	Physical attack (24)	Chloride attack (10)	Biological attack (19)
	Frost action (139)	Initiated chloride attack (8) Carbonation (6) Water (5) Alkali-silica reaction (ASR) (1)	

Table 5-2: Service condition as primary damage cause. The distribution of the secondary and tertiary causes of damage in the investigation

Service condition (650)		
Loading (483)	Abrasion (34)	Erosion (133)
Loading (183)		
Loading, shrinkage (178)	Abrasion, traffic (31)	Erosion (133)
Loading, traffic (60)	Abrasion, movement (2)	
Loading, temperature (25)	Other (1)	
Loading, overload (22)		
Loading, creep (8)		
Loading, movement of support (4)		
Other (2)		
Loading, earth pressure (1)		

5.2 Service life model

A simple method in predicting the development in time of the condition class (CC) for a given type of damage on a certain bridge element is introducing a linear trendline on the development of the average condition class. The condition put is that at the time of final inspection (age = 0) the average condition class (CC) was "0". This has been integrated in the BEA database and trendlines can be drawn for a number of search criteria.

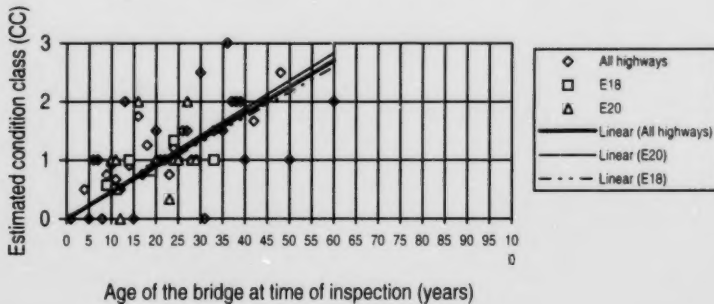


Fig 5c: Variation in time of the condition class development for the element "parapet fixing", all causes of damage for road section E18 Köping-Västerås (38 remarks), E20 Eskilstuna-Arboga (17 remarks) compared to all road sections (143 remarks).

6 Conclusions and further research

6.1 Relational database – A must in a modern bridge management system

The results presented so far with the help of the relational database BEA apply to the limited geographical areas. Data for bridges in big parts of the country are missing. At the present, the database is being completed with a further 100 bridges located on four new road sections, bringing the total number of investigated bridges to 370 on 23 different road sections. The relational database BEA ought to be updated with new inspections. Among other things, BEA today (1998-09-10) contains:

- 2898 Bridge inspections made between 1944 - 1995
- 2269 Major inspections
- 446 General inspections
- 105 Investigations
- 1044 Records from concrete castings
- 2165 Damages with stated type and cause
- 387 Chloride profiles

A relational database is today a must for an effective bridge management system. This relational database should be user-friendly and flexible. One of the obvious advantages using a relational database is the possibility to use a number of search criteria. In the Licentiate Thesis, "Konstbyggnaders reella livslängd", a number of different search criteria were used. It was discovered that five structural members are, together, responsible for most of the inspection remarks.

Table 6a: Damage distribution on certain structural members situated on 19 different road sections in the investigation

Road section	Edge beam (%)	Support (%)	Deck slab (%)	Slope and Embankment end (%)	Sum (%)
Arboga-Eskilstuna	19	11	10	12	52
Arboga-Örebro	31	8	9	7	55
Dorotea-Vännäs	16	2	0	18	36
Enköping-Bålsta	7	51	0	0	58
Eskilstuna-Arboga	19	17	12	13	61
Eskilstuna-Mariefred	19	18	18	9	64
Köping-Fagersta	27	0	9	7	43
Köping-Västerås	31	3	15	7	56
Nordmaling-Piteå	41	7	6	13	67
Nyköping-Stockholm	16	20	13	7	56
Ramnäs-Kopparberg	19	12	2	23	56
Storuman-gränsen	14	3	6	18	41
Strömsholm-Ramnäs	18	18	2	11	49
Strömsund-Arvidsjaur	24	7	5	20	56
Uppsala-Gävle	32	9	13	3	57
Västerås-Enköping	20	20	7	19	66
Örebro-Grängesberg	39	6	8	13	66
Örebro-Laxå	19	18	1	15	53
Östhammar-Karlholm	28	4	20	10	62

By changing the search criteria in BEA, new facts can be revealed. The percentage of damage remarks reported on certain structural members for certain construction types of bridges are presented in table 6-2. The following four types of construction were investigated:

- Beam and slab bridges (32 bridges),
- Beam and slab frame bridges (24 bridges),
- Frame bridges (162 bridges)
- Slab bridges (44 bridges)

The structural member "edge beam" are responsible for 24% of all inspection remarks in the investigation. The structural members "edge beam", "support", "deck slab" and "slope and embankment end" together answer for almost 50% of all inspection remarks, regardless of search criterion. Changes in the design, above all in details such as parapet fixing and support placing. More careful inspection of slopes, embankment ends and fillings at the final inspection should be taken into account.

Table 6b: The percentage contribution, from four structural members, to the inspection remarks on four different types of construction.

Type of construction	Edge beam (%)	Support (%)	Deck slab (%)	Slope and Embankment end (%)	Sum (%)
Beam and slab bridge	25	13	4	6	48
Beam and slab frame bridge	24	6	5	19	54
Frame bridge	25	12	11	11	48
Slab bridge	24	12	10	11	57

6.2 Bridge inspections

The cause of damage "environmental action" is highly overrated as a primary cause. The cause of damage "defective construction" has not been used, as a primary cause in the correct extent. The cause of damage "defective design" has not been used, as a primary cause, in the correct extent.

The extent of the final inspection as well as the physical measuring points should be integrated in valid codes and regulations. This should be even considered for the guarantee inspections.

6.3 Service life models

When assessing the service life for a structural member in a certain service life condition, a lot of search criteria is needed. It is also important to note that the service life of bridges often depends on the condition of its different structural members. The deterioration of one can inevitably start a deterioration process on one or more other structural members - The domino effect.

6.4 Future research

The R&D project continued through 1998 by increasing the number of bridges in the investigation to 370. Even the relational database BEA has been further developed for new search criteria.

Several service life models have been taken into consideration. The author will present the results in "The real service life of Swedish Road Bridges" during 1999.

7 References

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7 PROPERTIES OF, AND SERVICE LIFE PREDICTIONS FOR, HIGH PERFORMANCE CONCRETE IN TRANSPORTATION STRUCTURES

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Abstract

In a number of recent major projects, owners have required designers to demonstrate that the completed structure will provide an adequate service life in its given environment. In order to achieve this it is necessary to be able to model the deterioration process and determine the relevant properties of the concrete. For structures in chloride environments, the principal process governing the service life is the penetration of chloride ions through the concrete cover to the steel reinforcement. For transportation structures other long term performance parameters need to be defined; these include freeze-thaw durability, scaling resistance, abrasion resistance, strength, elasticity, shrinkage and creep. This paper compares a broad range of physical properties and time to corrosion characteristics of two concrete mixes. The high performance concrete is a 60-MPa air-entrained mix using St. Lawrence Cement Type 10E-SF blended silica fume Portland cement and ground, granulated blast furnace slag as used in the fabrication of the precast concrete tunnel linings for the Toronto Transit Commission Sheppard Line Subway (1996 & 1997). The reference concrete is a 30-MPa air-entrained mix using Type 10 Portland cement and slag typical of Ontario bridge deck concrete (other than post tensioned construction). Service life predictions were carried out using laboratory determined transport coefficients for a single simulated case (e.g. exposed reinforced concrete bridge deck with periodic application of de-icing salts). A computer program that models the flux of chloride due to combined transport mechanisms was used for the predictions. The work demonstrates the superior potential service life characteristics of the high performance concrete compared to typical current bridge deck concrete due mainly to the very low chloride transport properties of the 60-MPa concrete. Life cycle cost estimates indicate significant economic advantages of the high performance concrete for transportation structures, e.g. highway bridge decks in chloride contaminated freeze-thaw environments.

Keywords: chloride diffusion, high performance concrete, life cycle costs, permeability, service life, , transportation structures.

1 Introduction

The introduction of high performance concrete (HPC) in the 1980s represented a further step in the evolution of this the most universal and widely used construction material. While the initial commercial application of HPC was mostly based on its superior mechanical properties such as higher compressive strength (Ryell and Bickley 1987) the greater expected durability of HPC has lead increasingly to its adoption for structures built to withstand aggressive environments. Notable Canadian examples include the Hibernia Offshore Platform, Newfoundland (Hoff 1998) and the Confederation Bridge Crossing between New Brunswick and Prince Edward Island (Tadros 1998). In addition a number of Canadian and U.S. highway agencies and municipalities such as the Quebec Ministry of Transportation (Coulombe and Vézina 1998) are now using high performance concrete in the construction of new bridges and in the rehabilitation of existing ones.

Construction and testing of the 5,550 rings of precast concrete tunnel lining for the Toronto Transit Commission (TTC) Sheppard Line Subway (Hart et al 1997) between February 1996 and April 1997 demonstrated that very demanding specification requirements covering chloride transport rates in concrete can be met using HPC produced at a state-of-the-art manufacturing plant. Specifically chloride diffusion coefficients ranged from 313 to $1300 \times 10^{-12} \text{ m}^2/\text{sec}$ and most rapid chloride permeability values (ASTM C1202) were less than 500 coulombs. In service life modelling for reinforced concrete structures in a chloride environment chloride diffusion values of the concrete are the main criteria in determining time to corrosion of the near surface steel that would precipitate the first maintenance repairs to the structure.

The precast concrete tunnel lining project for the TTC subway is an outstanding example of the use of the superior durability related properties of high performance concrete. The segments for the 5.2 meter internal diameter tunnel were produced at a new 6,000 m² state-of-the-art manufacturing plant near Guelph, Ontario by a joint venture partnership consisting of St. Lawrence Cement Ltd., Ontario, Con Cast Pipe Inc., Ontario and Concrete Systems Inc., Hudson, New Hampshire. The type 10E-SF blended silica fume Portland cement used in the precast tunnel segments was the same product used for the concrete gravity structure in the Hibernia Offshore Platform (Hoff 1998).

The success in meeting the very demanding concrete quality specification requirements for the tunnel lining lead to St. Lawrence Cement sponsoring the study reported in this paper which explores the application of high performance concrete containing Type 10E-SF silica fume Portland cement in typical bridge deck construction.

The objective of this test program is to demonstrate that high performance concrete containing Type 10E-SF silica fume Portland cement, such as that produced for the TTC tunnel segments, has a standard of long term durability significantly greater than, and a life cycle cost considerably less than, current Ontario bridge deck concrete.

In this test program the TTC tunnel segment concrete (referred to as 60-MPa HPC) was compared to a 30-MPa air entrained concrete typical of concrete supplied for Ontario bridge decks other than post tensioned construction (referred to as 30-

MPa bridge deck concrete). The test program on the two types of concrete included freeze-thaw durability, scaling resistance, abrasion resistance, chloride ion penetration, compressive strength, modulus of elasticity, shrinkage and chloride diffusion.

The characteristics of the 60-MPa high performance concrete were also compared with the Federal Highway Administration (FHWA) definitions, using long-term performance criteria, for various grades of high performance structural concrete (Goodspeed et al 1996).

2 Materials and test procedures

For the 60-MPa HPC, except for the shrinkage and standard cylinder tests, testing was carried out on specimens drilled from, or cut from, concrete blocks cast during the 1996 production of the tunnel segments. These blocks of dimensions 635 mm x 285 mm x 200 mm deep were cast specifically for the specified production testing due to the difficulties of coring the segments without cutting the reinforcing steel. During production the test blocks were stacked on the carousel production line adjacent to the segments (Hart et al 1997).

The 60-MPa HPC blocks used in the testing program were specimens cast on October 1, 1996, October 10, 1996 and October 24, 1996 respectively. The shrinkage and standard cylinder specimens were fabricated from laboratory mixed concrete using similar materials and mix proportions to the production segments, see table 1.

For the 30-MPa bridge deck concrete comparison mix all test specimens were fabricated under laboratory conditions from a 3m³ load of ready mixed concrete delivered to the Trow Brampton laboratory on August 26, 1997. Concrete mix proportions and fresh concrete properties are summarized in table 1. Three concrete blocks, dimensions 635 mm x 285 mm x 200 mm deep, were fabricated from the ready mixed concrete for core and prism samples. There are then significant differences in the age, curing and hence maturity of some of the test specimens representing the two types of concrete. Essentially the concrete blocks from the 60-MPa HPC cast in October 1996 were subject to a 7 1/2 hour, 55°C steam curing process followed by 5 days in a moisture chamber at 100% relative humidity. Until cutting and coring of the specimens in July/August 1997 the blocks were stored in Trow's yard exposed to the elements. In contrast most specimens representing the 30-MPa bridge deck concrete were tested after 56 days laboratory moist curing following fabrication or cutting/coring from the test blocks.

This longer than normal period of moist curing for the 30-MPa bridge deck concrete is recommended by Goodspeed et al, 1996; it also results in concrete specimens with a degree of maturity closer to that of the 60-MPa HPC blocks cast in October 1996.

The schedule of tests is presented together with a summary of test results in table 2.

With the exception of the chloride diffusion coefficient test methods were ASTM standards and procedures from the Ministry of Transportation, Ontario (MTO) laboratory standard as noted in table 2.

Test procedures used to determine the chloride diffusion coefficient of the concrete core samples are described by Hart et al, 1997. The test protocol, based on the work of Wood et al, 1989, uses an elevated temperature (40°C) and a highly concentrated salt solution (5M NaCl) to accelerate the penetration of chlorides into the concrete samples. At the conclusion of 40 day, 80 day and 120 day immersion periods precision profile grinding techniques (1 mm increments) and chemical analysis are used to establish a chloride profile of the surface layer of concrete. Fick's second law of diffusion along with some assumptions for boundary conditions are used to determine the diffusion coefficient by curve fitting the chloride profile.

3 Test results and analysis

The concrete mix proportions for the 60-MPa HPC and the 30-MPa bridge deck concrete are presented in table 1. Concrete tests on the two types of concrete are summarized in table 2. The mix proportions shown for the 60-MPa HPC are for the laboratory mixed concrete used to fabricate the concrete prisms for the shrinkage test and the standard compressive strength cylinder specimens. Similar concrete mix proportions were used in the production concrete for the tunnel segments.

The ready mixed concrete used to represent the bridge deck concrete was supplied by the Dufferin-Custom group (Division of St. Lawrence Cement) to meet the requirements of the Ministry of Transportation, Ontario for 30-MPa bridge deck concrete.

Besides the use of Type 10E-SF Portland silica fume cement in the 60-MPa HPC in a quantity 15% greater than the Type 10 normal Portland cement used in the 30-MPa bridge deck concrete there are other important differences between the two types of concrete. The lower water content of 60-MPa HPC is largely due to the addition of high amounts of the high range water reducing admixture and the air content of the 60-MPa HPC is significantly lower than that of the 30-MPa bridge deck concrete.

The rapid freeze-thaw characteristics and the 100 cycle salt scaling resistance of the two types of concrete were acceptable. It should be noted that the air content of the 60-MPa HPC tunnel segment production concrete was below the CSA A23.1 requirements for concrete subjected to freeze-thaw conditions and that tests during production using the ASTM C457 procedures indicated an air void spacing factor consistently higher than considered acceptable by the CSA standard. It is clear though that the 60-MPa HPC concrete has a very high freeze-thaw durability as measured by the two series of demanding tests and that the low air content/high spacing factor has not adversely affected durability of the concrete.

Table 1: Concrete mix proportions (kg/m³) and tests on plastic concrete

Materials and Properties	60-MPa HPC	30-MPa Bridge Deck Concrete
Type 10 Normal Portland Cement (St. Lawrence Cement)		265
Type 10E-SF Portland Silica Fume Cement (St. Lawrence Cement)	305	
Granulated Slag (St. Lawrence Cement)	145	90
Fine Aggregate	929	700
20 mm Coarse Aggregate	953	1,100
Water	137	155
TOTAL	2,469	2,310
High Range Water Reducing Admixture	4.49 litres	1.07 litres
Air Entraining Admixture	0.14 litres	0.1 litres
Slump	95 mm	80 mm
Air Content	3.4%	6.2%
Mass Density	2,460 kg/m ³	2,334 kg/m ³
Concrete Temperature	22°C	24°C
Water/Cementitious Ratio	0.30	0.44

Table 2: Summary of concrete tests

Concrete Characteristic	Test Method	60-MPa High Performance Concrete	30-MPa Bridge Deck Concrete
Resistance to Rapid Freezing & Thawing (314 Cycles)			
Relative Dynamic Modulus of Elasticity (%)	ASTM C666	92.2	95.4
Length Change (%)		0.005	0.005
Salt Scaling Resistance (100 Cycles)			
Visual Rating	ASTM 672	1	1
Loss (kg/m ²)	MTO LS412	0.24	0.26
Resistance to Chloride Ion Penetration			
Coulombs passed in 6 hours	ASTM 1202	48	1,502
Chloride Diffusion Coefficient (x10⁻¹⁸ m²/s)			
40 days	ASTM 1202	17	1,335
80 days	See text	545	3,310
120 days		398	2,650
		331	2,010
Cylinder Compressive Strength (MPa)			
3 days		40.2	25.1
7 days	ASTM C39	52.2	32.1
28 days		62.4	40.0
56 days		68.9	44.9
Core Compressive Strength (MPa)			
Static Modulus of Elasticity (GPa)	ASTM C469	67.4	46.1
Abrasion Resistance			
Loss in Mass (gms)	ASTM C944	44.8	32.0
Depth of Wear (mm)		1.03	2.43
Shrinkage			
Length Change (%) 28 days	ASTM C157	<0.5	<1.0
Length Change (%) 56 days		-0.022	-0.019
Length Change (%) 180 days		-0.028	-0.026
		-0.048	-0.039

The differences in compressive strength and modulus of elasticity between the two concrete mixes from 3 days to 56 days are normal considering the significantly

* Values for Chloride Diffusion Coefficient noted in brackets represent specimens where the sides of the core samples were coated with epoxy resin to ensure one dimensional flow of the chloride solution into the concrete.

higher water to cementitious materials ratio of the 30-MPa bridge deck concrete. The comparison of compressive strength between the 60-MPa HPC standard cylinders after 56 days moist curing and the 10 month core specimens is interesting in that the strength difference is only approximately 2% of the mean value. In essence then the maturity/age/curing characteristics that influence strength gain are similar for the older steam cured production concrete compared to the much younger laboratory mixed concrete specimens exposed to 56 days of moist curing.

The ASTM C944 abrasion resistance test that gives an indication of relative wear resistance using a rotating cutter mounted on a drill press showed superior qualities for the 60-MPa HPC however the depth of wear on both types of concrete was very small.

Length change measurements of concrete prisms stored at room temperature and 50% relative humidity for 180 days show no unusual characteristics. The drying shrinkage of the 60-MPa HPC was slightly greater than the specimens representing the 30-MPa bridge deck concrete.

The widely used rapid chloride permeability test (ASTM C1202) that estimates chloride permeability by measuring the electrical conductivity of the concrete and provides definitions of the degree of chloride ion penetrability based on a unit of charge passed in the test (coulombs) classifies the two types of concrete as follows:

60-MPa HPC	: Negligible Chloride Ion Penetrability
30-MPa Bridge Deck Concrete	: Low Chloride Ion Penetrability

The more rigorous chloride diffusion test based on Fick's second law of diffusion using 40 day, 80 day and 120 day chloride immersion periods shows striking differences in chloride penetration characteristics for the two types of concrete. As far as is known to the authors the chloride diffusion values for the 60-MPa HPC, i.e. less than $600 \times 10^{-15} \text{ m}^2/\text{sec.}$, are amongst the lowest values reported for any production concrete and represent a very high resistance to the penetration of chloride ions. In contrast chloride diffusion values for the 30-MPa bridge deck concrete, i.e. generally between 2,000 and $3,000 \times 10^{-15} \text{ m}^2/\text{second}$, as will be shown later, are not adequate under many conditions to prevent the penetration of chloride ions through the concrete cover to the steel reinforcement for the normal anticipated service design life of most transportation structures.

4 Comparison with FHWA performance characteristics

Goodspeed, et al, 1996, have defined for the U.S. Federal Highway Administration (FHWA) a number of grades of high performance concrete (HPC) for various exposure conditions. FHWA's primary purpose in preparing a definition for HPC is to stimulate the use of higher quality concrete in highway structures. A number of technical parameters, including four related to deterioration resistance, have been identified as sufficient to represent HPC long term performance.

The FHWA Grade 3 HPC has the most stringent requirements for the deterioration related performance characteristics; such criteria can be applied to bridge deck concrete exposed to the most severe conditions including large amounts

of de-icing salts and heavy traffic volumes with studded tires allowed. The 60-MPa HPC as used in the tunnel segments meets all the durability requirements for the Grade 3 HPC. The 30-MPa bridge deck concrete fails to meet the Grade 3 HPC requirements for abrasion resistance and chloride penetration.

5 Service life predictions

Service life predictions were determined using a PC-based program which takes into consideration multi-mechanistic transport processes (diffusion, convection, sorption and wicking) and accounts for the temperature and time dependence of the relevant transport coefficients. The model also allows for non-linear chloride binding and changes in the surface concentration.

The analysis presented here is intended to represent the ingress of chlorides into a concrete bridge deck exposed to the periodic application of deicing salts. For this purpose it was assumed that the predominant transport mechanism was diffusion and that the diffusion coefficient varied with time according to the following relationship:

$$D_t = D_{28} \cdot \left\{ \frac{t_{28}}{t} \right\}^m$$

Where: D_t = diffusion coefficient at time t

D_{28} = diffusion coefficient at time $t_{28} = 28$ days

m = constant

The relationship is consistent with similar mathematical descriptions proposed by other investigators (Mangat and Molloy 1994; Maage et al 1995)

Values of D_{28} and m were selected as input variables for the model to 'best-fit' the experimental chloride diffusion data and to be consistent with published values for m (Mangat and Molloy 1994; Maage et al 1995; Bamforth 1998), the following values were used:

	HPC	30-MPa
$D_{28} \text{ (m}^2\text{/s)}$	6×10^{-13}	3.5×10^{-12}
m	0.35	0.35

The surface concentration was input as a time-dependent function, such that the chloride content increased linearly at a rate of 0.1% *Cl* by mass of concrete up to a maximum of 1.0% at 10 years; this is reasonably consistent with field data.

Figure 1 shows predicted chloride concentration profiles for the two different concrete mixes after 20 years exposure. Also shown is a theoretical chloride threshold level, $C_t = 0.05\%$ (by mass of concrete), which is defined as the level of chloride required to initiate corrosion. The profiles are very different for the two concretes and it can be seen that the threshold level has penetrated to a depth of approximately 50 mm in the standard 30-MPa concrete and only around 20 mm in the high-performance concrete. This prediction infers that corrosion of steel reinforcement with a cover depth of 50 mm will be initiated after just 20 years for the 30-MPa concrete. This

doesn't necessarily imply the end of the service life, but does indicate that some level of rehabilitation will be required between about 20 and 25 years. This is not inconsistent with past experience in Ontario.

Figure 2 shows the rate of penetration of the chloride threshold ($C_t = 0.05\%$) for the two concretes. After 50 years the threshold has penetrated only 30 mm into the HPC compared with more than 70 mm in the 30-MPa concrete. Similarly, after 100 years the depths of penetration are 40 mm and 95 mm in the high-performance and 30-MPa concrete respectively. If a concrete bridge deck was designed in accordance with Canadian Standards Association CSA A23.1 the minimum depth of cover should be 60 mm in this exposure condition. If this minimum cover were achieved the model would predict a corrosion initiation period of 30 years in the 30-MPa concrete and somewhere in the region of 300 years for the HPC!

6 Life cycle costs

There are increased costs associated with the use of HPC due to the use of higher cement contents, silica fume and relatively high doses of superplasticizer. It is not possible to give precise cost differences as prices will vary with each application depending on (among other things) the actual mix composition and quantities required, the location of the job and the proximity of plants capable of supplying HPC. However, in the Ontario market place it is likely the HPC mix described here would cost anywhere between 25% to 50% more than the 30-MPa concrete.

It is also not possible to generalize about life cycle costs, as these will vary tremendously between specific applications. However, it is possible to develop a hypothetical case to demonstrate how the increased cost of HPC might affect the costs over the whole life of a structure. For instance, if we assume that the HPC costs \$48/m³ more than 30-MPa concrete, then the increased cost of a 250-mm thick HPC bridge deck would be only \$12/m² of deck area. This price differential might actually be less if the savings due to the possible omission of waterproofing and an asphalt-wearing course were considered for the HPC deck. However, there are likely to be some increased costs associated with the use of HPC due to more stringent requirements regarding quality control, placing, finishing and curing. If we consider a 75-year life cycle no major rehabilitation of the HPC deck should be envisaged provided that a reasonable cover depth to the steel is ensured and other construction defects (e.g. excessive shrinkage cracking) are avoided.

The service life analysis presented here together with previous experience indicates that significant repairs would be necessary for a 30-MPa concrete bridge deck and that these would be associated with periodic replacement of the joints, repair of delaminated concrete and replacement of the waterproofing and asphalt layer. The cost associated with a major rehabilitation involving an overlay, waterproofing and paving vary like other costs, although values in the region of \$300/m² are often used for analysis. The present value of these future costs is highly sensitive to the time of the repair and the discount rate assumed for analysis (i.e. to convert future costs to present value). Figure 3 shows the relationship between the present value and the time at which the repair is carried out for different discount rates. This figure assumes a

one-time repair only; values would be increased somewhat by additional repairs at later ages. Also shown in this figure is the \$12/m² premium for using HPC.

It is evident from Figure 3 that for most scenarios the increased costs associated with the future repairs of the 30-MPa concrete significantly exceed the higher initial cost of using HPC. The analysis presented here is simplistic, but is in agreement with more detailed financial analyses presented for specific projects involving HPC (Fletcher 1997).

7 Summary and conclusions

Since the early 1960's it has been recognized that the premature deterioration of concrete bridge decks in many parts of North America characterized by surface spalls, potholes and cracking of the concrete, is due to the corrosion of the steel reinforcement in a chloride laden environment.

The development of high performance concrete over the last 15 years has demonstrated the potential of this material to provide greatly increased concrete durability in the aggressive environment of a bridge deck subjected to frequent applications of de-icing salts.

The work reported in this paper clearly demonstrates the striking difference in the chloride penetration characteristics between typical 30-MPa bridge deck concrete using normal portland cement and 60-MPa high performance concrete containing portland silica fume cement. The 30-MPa concrete as tested was produced at a typical ready mix concrete plant; the 60-MPa HPC was produced at a state-of-the-art pre-cast concrete plant set up for the production of precast concrete tunnel linings.

Time to corrosion modelling for the two types of concrete indicate a corrosion initiation period, assuming adequate concrete cover to the top steel, significantly greater than the normal design life of a highway bridge for the 60-MPa high performance concrete. In contrast the typical 30-MPa bridge deck concrete is forecast as requiring some level of rehabilitation between about 20 and 25 years.

The increased cost of the high performance concrete estimated at \$12 per square meter of bridge deck is small compared to the initial cost of the deck superstructure estimated to be about \$1,000 per square meter and a major deck slab rehabilitation effort estimated at \$300 per square meter.

It is concluded that there are significant economic advantages in selectively using high performance concrete in highway structures; bridge decks are the prime example.

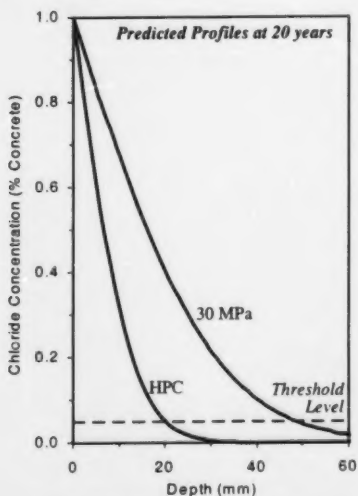


Fig. 1: Effect of concrete quality on predicted chloride profiles at 20 years

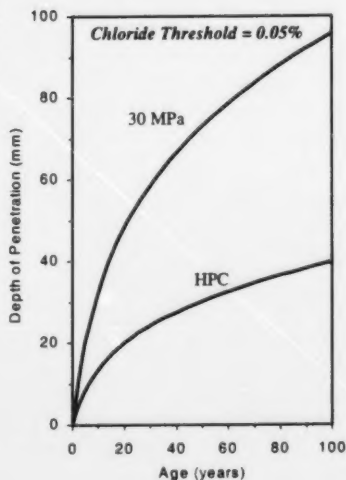


Fig. 2: Effect of concrete quality on the rate of chloride penetration

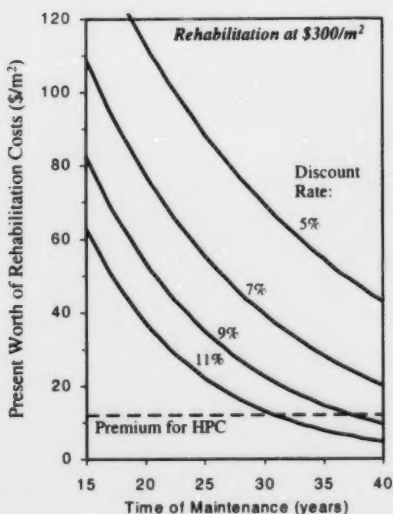


Fig. 3: Increased life cycle costs due to one-time maintenance

8 Acknowledgments

The work reported in this paper was funded by the St. Lawrence Cement Inc., Ontario.

Fabio Fregonese, Laboratory Supervisor, Trow Consulting Engineers Ltd. was responsible for the laboratory testing program. His valuable contribution and assistance is acknowledged.

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8 SERVICE LIFE PREDICTION OF REINFORCED CONCRETE STRUCTURES, BASED ON IN-SERVICE CHLORIDE PENETRATION PROFILES

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Abstract

Chloride ingress is one of the most important causes of corrosion of the reinforcement in concrete structures such as on-shore constructions in marine environment. Those structures can be protected by means of a protective coating or by a water-repellent agent. Based on experimental results, it was decided in 1993 to apply IBTEO (isobutyl-tri-ethoxy-silane) on the quay-wall of the new container terminal at Zeebrugge harbor, Belgium. Two subsequent research programs were executed to evaluate the effectiveness of the water-repellent treatment. The first test program took place after 3 years of in-service exposure (1996) and the second one after 5 years (1998). These research programs provide one of the first mid-long term records concerning the effectiveness of a water-repellent agent under in-site circumstances. A probability method for interpretation of test results and prediction of the service life of the quay-wall was updated. The relevance of the mathematical model, the effectiveness of the protective system and the increase of durability of the concrete structure will be discussed.

Keywords: chloride penetration, water-repellent, service life prediction, reliability, chloride penetration, corrosion

1 Introduction

Especially for off-shore and marine constructions, the exposure to chlorides from sea water and marine air are amongst the most important actors in the corrosion process of concrete reinforcement rods. The ingress of chloride ions into the pores of the concrete is caused by diffusion through the pores if they are filled with water or capillary suction if the pores are empty. When the chloride ions reach the reinforcement bars, the passivating oxide layer may be depassivated, which initiates the local corrosion of the rods (Kissel and Pourbaix 1996; Schueremans and Van Gemert 1997).

The chlorides in the concrete either come from the components of the fresh mix or from external contamination. The chlorides that stand for the corrosion risk of the reinforced concrete structure are the free chlorides in the pore water and the water soluble salts (Salt of Friedel) that act as a stock of free chlorides. In order to increase the durability of the concrete, it was decided to apply a hydrophobic treatment on the quay-wall of the new container terminal at Zeebrugge, Belgium.

The construction of the container terminal at Zeebrugge was ordered by the Ministry of the Flemish Community, Sea-Harbor Division and finished in 1993. Figure 1 gives a general view of the quay-wall, constructed on top of cylindrical sunk down reinforced concrete cells, so-called caissons. For ease of application it was decided to apply a hydrophobic treatment with highly concentrated solvent-free compounds based on iso-butyl-tri-ethoxy-silan, in order to prevent it for damage caused by chloride penetration, pitting corrosion and alkali-aggregate-reaction.

By means of a preliminary research program (Fliedner 1995; Schueremans et Van Gemert 1997), the effectiveness of the hydrophobic agent was evaluated. The production, application and working mechanism of silanes are described in detail elsewhere (Schueremans et Van Gemert 1997).

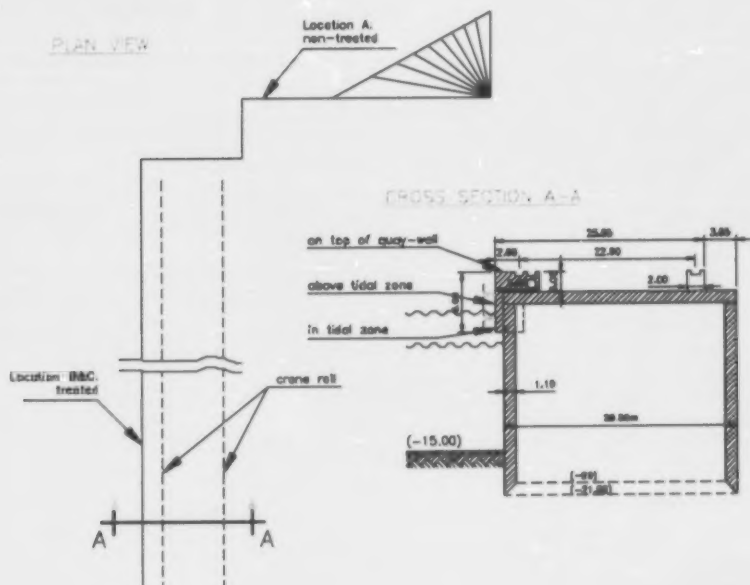


Fig. 1: Plan view and cross section of the quay-wall and locations of the cores

To evaluate the risk caused by chloride ingress, service life predictions are very important, both for existing and planned structures (Clifton 1993). The time dependent reliability method, as used and updated in this paper, is based on the chloride profiles of the free chlorides as measured at different locations on the container quay-wall and at different depths from the concrete surface. Therefore cores were taken on locations treated with the water repellent agent as well as on non-treated locations. These cores were sawn in slices of 1 cm thickness. For each slice of the concrete cores where the chloride concentration is measured, a value of the diffusion coefficient D_i can be back-calculated using the inverse relationship of Fick's 2nd law. These values are used to estimate the probability density function of the diffusion coefficient D at the different locations. Once the probability density function D is estimated, a reliability analysis is performed to calculate the service life prediction value. This enables to evaluate the effect of a preventive hydrophobic treatment.

2 Test Program

In 1996, a first in-service evaluation of the performance of the water-repellent agent was performed (Van Gemert et Schueremans 1996a; Van Gemert et Schueremans 1996b) on cylindrical cores taken at different locations in the quay-wall (see fig. 1): treated (location 2) and non-treated (location 1) locations, in the tidal zone, above water level and on top of the quay-wall.

Besides the penetration depth and porosity, the following material properties were measured at three different depths (0-9 mm, 11-20 mm, 40-60 mm): pH, carbonation depth, compressive strength, cement content and chloride content. The characteristics relevant for this paper are summarized in table 1. A full report can be found in earlier papers (Van Gemert et Schueremans, 1997).

In 1998, a similar test program was executed to re-evaluate the effectiveness after 5 years of in-service exposure, table 2. Cores were taken near the ones taken in 1996. The chloride concentration was measured at several depths. To obtain better estimates of the diffusion coefficient, the chloride concentration was measured at depths slightly different from the former test program. To increase the reliability, some additional analyses were added. To obtain comparable values, the amount of cement as measured in 1996 was used to transform the chloride content per mass of concrete into the chloride content per weight of cement.

Figure 2 and 3 show the water soluble chloride profiles as they are obtained at two moments in time. The chloride contents were determined by means of wet chemical analysis, according to the Belgian Standard NBN B15-257. The chloride content, obtained by wet chemical analysis, equals the amount of free chlorides and a great deal of the chlorides, bound under the form of the Salt of Friedel which dissolves in the water during extraction. In fact, these water soluble chlorides mean the real danger for corrosion of the reinforcement (Van Gemert and Horckmans 1992).

The relation water soluble to acid soluble chlorides amounts 94% (1996) and 100% (1998) in the executed tests [Van Gemert and Schueremans, 1995, 1998]. Thus, the amount of insoluble chlorides is very restricted as the greater part of chloride loading is caused by the external contamination, i.e. the marine environment. It can be observed that the chloride content at the non-treated locations is significantly higher than in the treated locations.

Table 1: Material properties and water-soluble chloride content, 1996

	Core no.	n [v%]	Dry density [kg/m ³]	amount of cement [kg/m ³]	%Cl-/cem		
					x=4.5 mm	x=15.5 mm	x=50 mm
Non-treated location 1, fig. 1	A1	17.48	2157	284	2.77	3.44	2.18
	A2				2.07	3.22	1.84
	A3	14.57	2232		1.91	2.28	1.44
	A4	14.76	2224		3.35	2.36	1.51
Treated location 2, fig. 1	B5	15.66	2234	260	2.24	1.23	0.38
	B6				1.31	1.66	0.29
	B7	17.38	2193		1.75	0.94	0.32
	B8	16.70	2197		1.64	1.07	0.36
Treated on top of quay- wall	C9	14.86	2236		0.71	0.53	0.45
	C10				0.64	0.64	0.44
Mean value		16.00	2200	272			

Table 2: Water-soluble chloride content per weight of cement, 1998

	Core no.	%Cl-/cem					
		x=4.5mm	x=15mm	x=25 mm	x=45 mm	x=65 mm	x=95 mm
Non-treated location 1, fig. 1	A1	4.17			5.13		1.57
	A2	3.39			2.65		1.50
	A3	3.29			2.37		1.01
	A4	2.26	2.16	2.82	1.98	1.54	1.17
Treated location 2, fig. 1	B5	1.77			0.47		0.46
	B6	1.63			0.44		0.40
	B7	1.00			0.30		0.38
	B8	1.82	2.10	0.41	0.32	0.42	0.54
Treated on top of quay- wall	C9	0.48			0.42		0.60
	C10	0.40			0.55		0.51

The absence of an effective barrier, combined with the relatively high porosity, allowed the chlorides to penetrate very deep in the concrete in a relatively short time period. On the contrary, the chloride content in the treated zones is still at an acceptable level, and combined with the low carbonation depth, it presents no danger for reinforcement corrosion.

3 Service Life Prediction, a time dependent reliability analysis

Based on the measured material properties and chloride profiles, a service life prediction can be performed using a time dependent reliability analysis (Melchers 1987, Thoft-Christensen et al. 1996). The reliability analysis used in this paper is applicable to concrete deterioration associated with steel corrosion initiated by the action of chloride ions.

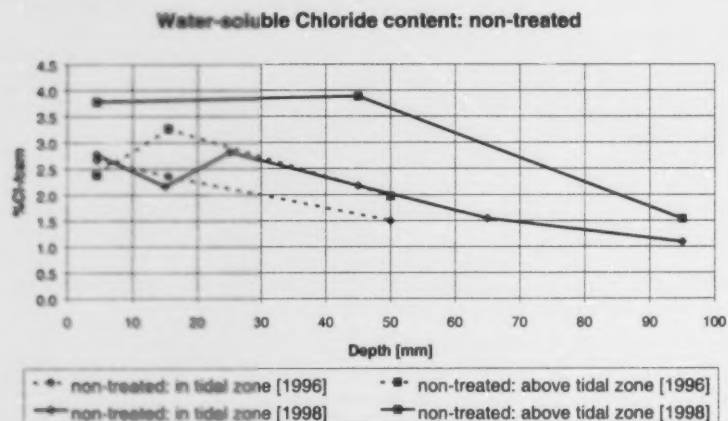


Fig. 2: Water-soluble chloride content by weight of cement at different depths in the cores, 1996 and 1998 (Location A).

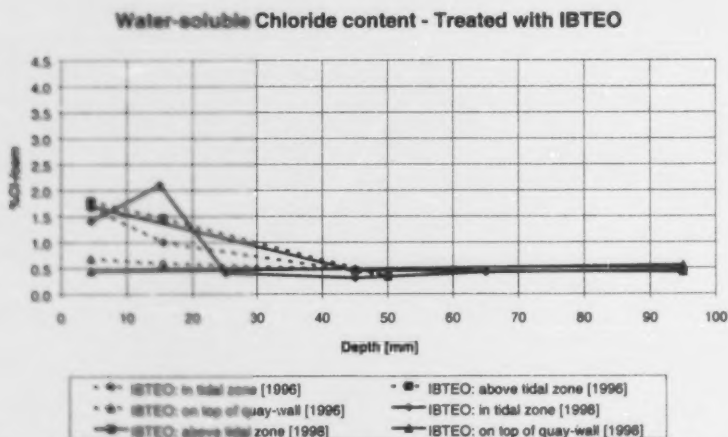


Fig. 3: Water-soluble chloride content by weight of cement at different depths in the cores, 1996 and 1998 (Location B and C).

Similar analyses can be performed for other deterioration processes as long as the mathematical formulation governing the failure mechanisms is available.

It is virtually impossible to introduce a mathematical model taking into account all the variables involved in the corrosion process. To estimate the service life of a given concrete element, many assumptions need to be made. To model the chloride transport process in a porous material, it is assumed that Fick's 2nd law applies, although it is a simplified representation of reality (Schueremans et Van Gemert, 1998, Prezze et al. 1996).

The diffusion law, Fick's 2nd law for one-dimensional chloride diffusion into concrete, takes the form :

$$\frac{\delta C}{\delta t} = \frac{\delta}{\delta x} \left(D \frac{\delta C}{\delta x} \right) \quad (1)$$

in which concrete is assumed to be a homogeneous, isotropic material. When assumed that no reaction occurs between concrete and chlorides, an explicit solution of this differential equation can be obtained, using the following boundary conditions:

- $C(x, t=0) = C_0$; $0 < x < \infty$ (the initial chloride concentration in the concrete mix) and
- $C(x=0, t) = C_s$; $0 < t < \infty$ (the chloride concentration loading at the surface of the concrete due to the marine environment),

$$C_i(x, t) = C_0 + (C_s - C_0) \operatorname{erfc} \left(\frac{x}{2\sqrt{Dt}} \right), \quad (2)$$

in which $C_i(x, t)$ is the amount of chlorides at time t at a distance x from the concrete surface.

A reliability analysis provides a means to evaluate the probability of failure of a component. The term component describes a structure or structural element whose limit state function is defined in terms of a single, continuous function known as the limit state function (Melchers 1987). In the present problem, where only the diffusion coefficient D is considered to be random, the limit state function $g(D)$ can be written as:

$$g(D) = C_T - C(D) \quad (3)$$

where C_T is the threshold chloride concentration and $C(D)$ is the chloride concentration at a distance x from the exposed surface at time t .

Assuming the diffusion coefficient has a lognormal distribution, the exceedence probability is consequently obtained as (Schueremans et Van Gemert 1998):

$$P_f = P(C > C_T) = 1 - \Phi \left(\frac{\ln(D_T) - \lambda_D}{\xi_D} \right) \quad (4)$$

where λ_D and ξ_D are the parameters of the lognormal distribution and $\Phi(D)$ is the standard normal cumulative distribution function.

With the chloride profiles of the concrete samples exposed during a given period, the diffusion coefficient can be back-calculated using an inversion of Fick's 2nd law (eq.2). In the inversion of Fick's second law, the following boundary conditions were used: $C_0 = 0.03\%$ Cl⁻ and $C_s = 7\%$ Cl/H₂O (Schueremans et Van Gemert, 1996).

Table 3 gives the mean value $\mu(D)$ and the standard deviation $\sigma(D)$ as well as the coefficient of variation $\text{cov}(D) = \sigma(D)/\mu(D)$ for the groups of cores taken at different locations for 1996. It is remarkable that the diffusion coefficient of the non-treated locations is nearly an order of magnitude higher than in the treated locations.

With the computed parameters of the lognormal probability distribution for the diffusion coefficient D , the service life can be predicted using the reliability analysis as outlined before. A corrosion initiation mode can be characterized in a simplified way by specifying a threshold chloride concentration beyond or above which corrosion would start. The reliability analysis is performed for the threshold chloride concentrations that equal 0.4 percent per weight of cement and 0.7 percent per weight of cement. As the free chlorides mean the real danger for corrosion, only the water soluble chlorides (table 1 and table 2) were taken into account.

For a conventionally taken probability of failure of 50 percent ($P_f = 0.5$) (Prezzi 1996), table 3 shows the time it will take in years for the threshold chloride concentration to reach the reinforcement. The service life prediction values (SLP) listed in table 3 assumed that the reinforcement is located at 120 mm from the exposed surface as could be measured from the taken cores. These values are based on the survey executed in 1996.

Table 3: Diffusion coefficients and service life prediction (SLP)-values for $P_f = 0.5$ at the different locations based on test programs after 3 years of in-service exposure

SLP-values For $P_f = 0.5$ at $x = 120$ mm	Test- Pro- gram	n_1	$\mu_1(D)$ [cm ² /sx10 ⁻⁸]	$\sigma_1(D)$ [cm ² /sx10 ⁻⁸]/ (cov(D))[%]	SLP [years] $C_T=0.4\%\text{Cl}^-$ /cem	SLP [years] $C_T=0.7\%\text{Cl}^-$ /cem
A (non-treated)	1996	12	9.64	7.0 / (72.5)	7.6	12.6
B (treated with IBTEO)	1996	12	1.18	1.0 / (88.1)	61	105.1
B (treated with IBTEO) on top of quay-wall	1996	6	1.32	1.7 / (128)	70	123.3

The values of the diffusion coefficient based on the chloride concentrations measured in 1998, after 5 years of in-service exposure are used to check the time-dependency of the diffusion coefficient, reflecting the influence of possible degradation of the hydrophobic agent in time. Table 4, left hand side, lists the parameters of the diffusion coefficient, back-calculated from chloride values measured at 5, 15, 25 and 45 mm depth. The survey executed in 1996 also contained chloride concentration values up to depths of 45 mm.

Moreover, the survey of 1998 is used to account for the influence on the diffusion coefficient of chloride concentration values, measured at higher depths (65 mm and 95 mm). Table 4, right hand side, lists the parameters as obtained from the full survey, including the values measured at 65 and 95 mm depth.

The method of hypothesis testing is used to calculate the significance of the time-dependency on the one hand and of the addition of values measured at higher depth on the other hand (Beirlant and Van Dyck, 1997).

The initial hypothesis in both cases reads:

$$H_0: \mu_1 = \mu_2 \quad (5)$$

Table 4: parameters of the diffusion coefficients based on the test program executed in 1998

Loc.	n_2	$\mu_2(D)$ [cm ² /s x 10 ⁻⁸]	$\sigma_2(D)$ [cm ² /sx 10 ⁻⁸]/ (cov(D))[%]	Ratio	n_3	$\mu_3(D)$ [cm ² /s x 10 ⁻⁸]	$\sigma_3(D)$ [cm ² /sx 10 ⁻⁸]/ (cov(D))[%]	Ratio
A	10	12.3	16.0 (130)	1.28	12	15.0	13.8 (92)	1.6
B	10	0.81	0.7 (91.5)	0.67	12	2.61	2.9 (111)	2.2
C	4	0.85	1.0 (115)	0.64	6	3.41	4.1 (119)	2.6
1998, depths: 5, 15, 25, 45 mm					1998, depths: 5, 15, 25, 45, 65, 95 mm			

The alternative hypothesis reads:

$$H_1: \mu_1 \neq \mu_2 \quad H_1: \mu_1 < \mu_3 \quad (6)$$

As the ratios in table 4 – right hand side – suggest an increase for the mean value of the diffusion coefficient adding chloride values measured at higher depth, a one-sided alternative hypothesis is used.

To assure sufficient proof in case the initial hypothesis is rejected, the level of significance is taken equal to 90 % in both cases:

$$\alpha = 0.10. \quad (7)$$

The test-statistic reads (Beirlant and Van Dyck, 1997):

$$T_0 = \frac{\bar{X}_1 - \bar{X}_2}{S_p \sqrt{\frac{1}{n_1} + \frac{1}{n_2}}}, \text{ in which: } S_p^2 = \frac{(n_1 - 1)S_1^2 + (n_2 - 1)S_2^2}{n_1 + n_2 - 2} \quad (8)$$

The initial hypothesis is rejected in case:

$$|T_0| > t_{\frac{\alpha}{2}, n_1 + n_2 - 2} \quad T_0 < -t_{\alpha, n_1 + n_2 - 2} \quad (9)$$

in which t_α is the proper quantile of the Student probability distribution function. These values are summarized in table 5.

Based on these hypothesis tests, one may conclude that the time dependency of the mean value of the diffusion coefficient is not significant. No significant proof can

be given of possible degradation of the water repellent agent after 5 years of in-service exposure.

On the other hand, adding values measured at higher depth (65 mm and 95 mm) does have a significant influence on the mean value of the diffusion coefficient. As the chloride concentrations at depth 65 and 95 mm are rather high, a remarkable high value of the diffusion coefficient is required to obtain these values based on diffusion as modelled by Ficks 2nd law.

Table 5: Overview of the hypothesis tests

Location	A	B	C	A	B	C
S_p	$9.58 \cdot 10^{-8}$	$8.9 \cdot 10^{-8}$	$1.47 \cdot 10^{-8}$	$4.64 \cdot 10^{-8}$	$2.26 \cdot 10^{-8}$	$1.21 \cdot 10^{-8}$
T_0	-0.266	0.990	0.490	-2.983	-1.634	-2.731
t_{α}	1.726	1.725	1.860	-1.316	-1.316	-1.372
Conclusion	keep H_0	keep H_0	keep H_0	reject H_0	reject H_0	reject H_0
Legend	Survey: 1998, depths: 5, 15, 25, 45 mm			Survey: 1998, depths: 5, 15, 25, 45, 65, 95 mm		

4 Update of SLP-values

As the time dependency is not-significant based on the executed surveys, the SLP-values are updated adding the values of the second survey to those of the first survey, thus accounting for the significant influence of the chloride concentrations as measured at higher depths. Table 6 lists these values. For ease of comparison with table 3, the SLP-values as obtained earlier based on the first survey alone are shown in between brackets.

Table 6: Diffusion coefficients and SLP-values for $P_f = 0.5$ at the different locations based on test programs after 3 and 5 years of in-service exposure

SLP-values For $P_f = 0.5$ at $x = 120$ mm	Test- Program	n_1	$\mu(D)$ [cm ² /sx10 ⁻⁸]	$\sigma(D)$ [cm ² /sx10 ⁻⁸]/ (cov(D))[%]	SLP [years] $C_f = 0.4\%Cl^-$ /cem	SLP [years] $C_f = 0.7\%Cl^-$ /cem
A (non-treated)	1996 + 1998	27	12.6	11.4 / (90.5)	6.2 (7.6)	10.4 (12.6)
B (treated with IBTEO)	1996 + 1998	27	1.98	23 / (118)	44 (61)	74 (105)
B (treated with IBTEO) on top of quay-wall	1996 + 1998	12	2.37	3.2 / (134)	42 (70)	70 (123)

5 Conclusions

Based on the test programs executed after 3 and 5 years of exposure in a real marine environment, a significant difference in diffusion coefficient is measured. This difference is caused by the influence of the chloride concentration values as measured at depths equal to 65 and 95 mm from the concrete surface. There was no significant influence of time on the increase of the diffusion coefficient, which could have been caused by possible degradation of the hydrophobic agent.

Although the significant increase of the diffusion coefficients result in lower estimates of the service life prediction values, the effectiveness of a hydrophobic treatment remains.

6 Acknowledgement

This work was conducted as part of a research program to examine the time dependent reliability of concrete structures. The execution of the 2nd test program was funded by Sivento NV. Their collaboration is gratefully acknowledged.

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9 AN ENGINEERED MODEL FOR SERVICE LIFE OF MARINE CONCRETE STRUCTURES

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Abstract

This paper presents a simple approach to predicting service life of marine reinforced concrete structures. This approach is based on the solution derived from Fick's 2nd law of diffusion with modifications. The modifications required are the corrections for chloride surface concentration C_s and chloride diffusion coefficient D . Acceptable steel corrosion rate is suggested for use in the prediction of service life.

Keywords: Service life, chloride penetration, corrosion

1 Introduction

Reliable prediction of service life of concrete structures is a complicated process. It requires intimate knowledge on the environments, material behaviours, degradation/interaction mechanisms and kinetics; and functional/safety requirements [Pommersheim and Clifton, 1985, Fagerlund, 1983]. Due to these complications, even with the narrowed list of variables, prediction of service life should be viewed as a qualitative process or at best semi-quantitative at this stage.

In a marine environment, concrete can be subjected to a range of chemical, physical, biological and electrochemical attack. Chloride induced corrosion of steel reinforcement – electrochemical attack – has been known to be the major cause of concrete deterioration. Hence, modelling of service life of reinforced concrete structures with respect to chloride induced steel corrosion has been the focal point of many research and studies. Such a model can be used in the design/materials selection/specification of new structures to meet specific service life requirements; in the assessment of residual service life of existing structures or in planning of maintenance/repair strategy. When used in combination with a life cycle costing

model, it can be used to formulate economical solution to combat reinforcement corrosion.

Service life of a concrete structure can be defined in different ways (Figure 1). Some of these definitions are:

- Time from construction until when the chloride content at the reinforcement is high enough to initiate steel corrosion (corrosion-free models);
- Time from construction to time at which signs of distress are visible/observable (acceptable corrosion models);
- Time from construction to time at which the member/component/structure is considered to be functionally unacceptable or unsafe (end-of-function models).

In this paper, an approach to the prediction of service life of concrete structures is given based on the acceptable corrosion rate (2nd definition). This concept is adopted based on the consideration that in practice, the initiation of 'real' maintenance/repair work is likely when signs of distress including rust stain, spalling are visible.

The basic modules needed are:

- Chloride penetration pattern vs time;
- Chloride threshold level;
- Steel corrosion rate vs time;
- Cumulative damage to concrete;

These items will be discussed in this paper. The objective of this paper is to present the rationale behind the process of prediction of service life of marine RC concrete structures. The key to predicting service life of marine RC structures is to evaluate the life of each component using an appropriate model/process. For a given structure, different models may be required for different components. The components with the shortest predicted service life would dictate the overall service life, especially when the failure of such a component has a serious structural/safety consequence. The discussion of service life prediction in this paper is therefore limited in scopes.

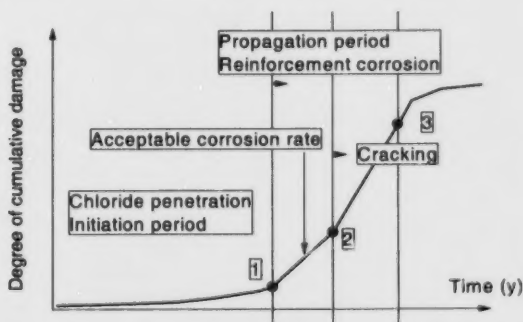


Figure 1: Possible "definitions" of service life

2 Prediction of chloride penetration into concrete

Fick's second law of diffusion (1) is commonly used in evaluating chloride penetration into concrete. For a semi-infinite slab with constant surface chloride concentration, a well-known solution can be found as follows:

$$\frac{\partial C}{\partial t} = D \frac{\partial^2 C}{\partial x^2} \quad (1)$$

$$C(x, t) = C_i + (C_s - C_i) \operatorname{erfc} \left(\frac{x}{\sqrt{4Dt}} \right) \quad (2)$$

where $C(x, t)$ is the chloride content at depth x at time t ; C_i is the initial chloride content of the concrete (usually negligible); C_s is the chloride content at the exposed surface; D is the chloride diffusion coefficient and erfc is the error function complement.

Based on this equation, the concrete is characterised by C_s and D ($C_i \sim 0$). Curve fitting programs are widely used to obtain values of D and C_s from an experimentally determined chloride profile at a known time. The $C(x, t)$ is predicted with relevant figures of concrete cover depth and exposure time. This procedure has been adopted in many models for predicting chloride penetration into concrete.

Recently, it has been pointed out that theoretically the use of Fick's law in the prediction of chloride penetration is not appropriate [Chatterji, 1994 a,b; 1995]. The main reason for this "inapplicability" is that Fick's law is valid for non-ionic diffusants through inert homogeneous medium. Chloride penetration into concrete is clearly an ionic transportation process involving other ions in the pore solution [Chatterji 1994c; Sergi 1992]. Concrete is an inhomogeneous and dynamic medium chemically and physically. Practically, analyses of concrete subjected to real marine environments indicate that C_s and D are not constants with exposure time [Johansen et. al 1995; Maage et. al 1996; Weyers et. al 1998]. From both theoretical and practical viewpoints, one must concede that the application of the solution derived from Fick's second law in the prediction of chloride penetration into concrete is not appropriate. The Nernst/Nernst-Planck equations are more appropriate for movement of ions [Chatterji 1994 a]. Even with these equations, catering specially for ionic transportation, the authors are not convinced that they can cope adequately with the chemical and physical variations of concrete with time. At this stage, it appears that the solution to Fick's 2nd law is still the most convenient and practical means of predicting chloride penetration into concrete, provided that adequate corrections are made.

With considerations to concrete behaviour and the nature of ionic movement, the parameters defining concrete in equation (2), C_s and D , can not be constants in the process of prediction of chloride penetration. C_s should be a time-dependent parameter and D should be dependent on time and depth of concrete. The time dependent corrections for C_s and D are necessary for the variations in pore structure and hydration/reaction between concrete and the environment, particularly in the

presence of chloride ions. The depth dependence of D is required for the concentration effect on ionic movement, which is related to both time and depth.

All of these needed "corrections" are based on empirical relationships with large degree of extrapolation in most cases. Hence even in the first step of making prediction service life, the qualitative nature of this exercise is fairly obvious. It should be noted further that empirical relationships are case- and environment-specific. The application of correction factors derived from empirical relationship requires care and understanding of relevant situations and collected data.

Linear and square root relationships have been suggested for time-dependent C_s build-up [Uli et al 1990; Zemajitis and Weyers 1996; Weyers 1998]. For the linear relationship, $C_s = C(0, t) = kt$ where k is a constant, a solution to Fick's 2nd law is:

$$C(x, t) = kt \left[\left(1 + \frac{x^2}{2Dt} \right) \operatorname{erfc} \left(\frac{x}{2\sqrt{Dt}} \right) - \left(\frac{x}{\sqrt{\pi Dt}} \right) e^{-x^2/4Dt} \right] \quad (3)$$

For the square root relationship, $C(0, t) = k\sqrt{t}$, $C(x, t)$ is given as

$$C(x, t) = k\sqrt{t} \left[e^{-x^2/4Dt} - \left(\frac{x\sqrt{\pi}}{2\sqrt{Dt}} \operatorname{erfc} \left(\frac{x}{2\sqrt{Dt}} \right) \right) \right] \quad (4)$$

D is assumed to be constant; C_i and $C(x, t)$ for large x are assumed to be zero in obtaining the above equations. The value of k can be estimated from a surface chloride concentration determined after a known time of exposure.

Since the build-up of surface chloride concentration is more likely and more significant in the vicinity of the splash zone and in air-borne chloride situation, equations (3) and (4) are more suited for the purpose of predicting chloride penetration in these situations. Equation (2) is more suited for submerged applications. The build-up of near surface chloride concentration with time is a local parameter and tends to reach a limiting value depending on the concrete and application [Weyers 1998; Bamforth 1993]. Since there is no existing method of estimating these limits, the following estimation procedures are proposed:

- Assume that concrete porosity at the surface is saturated with chloride ion (~35% NaCl equivalent). This is about 190 kg of Cl per m³ of saturated salt solution.
- Some chloride ions are bound in hardened cement pastes, which should be related to the binder content of the concrete (B). A binding capacity of 2.5% by weight of binder is proposed [Nagataki et al 1993]. The bound chloride is estimated as $2.5 \times B/100$. For a concrete containing 400 kg of binder, the amount of bound chloride is about 10 kg.
- For concrete with surface porosity P (eg. 10% or 0.1) and density ρ (eg. 2400 kg/m³), the surface chloride concentration C_s (total Cl concentration by weight of concrete) can be estimated as (bound chloride + $190 \times P$) / ρ . With the values given as example; C_s is estimated as $(10 + 19)/2400 = 1.2\%$ by weight of concrete. (The 'connection' between C_s to porosity can be used to cope with cracked concrete).
- Rough estimates of "limiting" C_s values (by weight of concrete) are about 1.5% for 32 MPa; 1% for 50 MPa and about 0.8% for 60 MPa (uncracked) concretes.

The above estimated values of C_s can be used in Equation (2) for long term prediction of chloride penetration and in Equations 3 and 4 as limiting values for kt or $k\sqrt{t}$. It is expected that in practice, due to salt recrystallisation, much higher value of C_s may be observed. However, salt crystals eg. NaCl do not participate in the inward movement of saturated Cl solution. Hence the use of these apparently high C_s values in the prediction of chloride penetration is not justified.

D is assumed to be constant in the above discussion. As noted previously, D is a time and depth dependent parameter. Provided that other physical and chemical processes do not cause damages to concrete, the measured chloride diffusion coefficient D should decrease with exposure time. This is partly due to the modification of the pore structure and partly due to the reactions between cement hydrates and advancing chloride ions. Chloride binding capacities of cementitious materials and pore refining/degree of hydration of the hardened cement pastes are time dependent. In practice, the trend of reduction of D with exposure time is well published [Maage et. al 1996; Mangat and Molley 1995; Tumidajski et. al 1995].

An empirical form of correction for the time dependence of D is $D_a t_a^\beta = K$ [Maage et. al 1996], where D_a is the chloride diffusion coefficient at time t_a and K is a constant. Chloride diffusion coefficient D_b at time $t_b > t_a$ can be estimated from D_a , and D_b is smaller than D_a . β is a parameter depending on the material, mix design and environment. In most if not all applications, β is not known. The reduction of D with time, therefore should be determined by obtaining two pairs of D and t and by plotting D vs t on a double log system. A straight line is expected and can be used in the extrapolation of D to the required exposure time as shown in Figure 2.

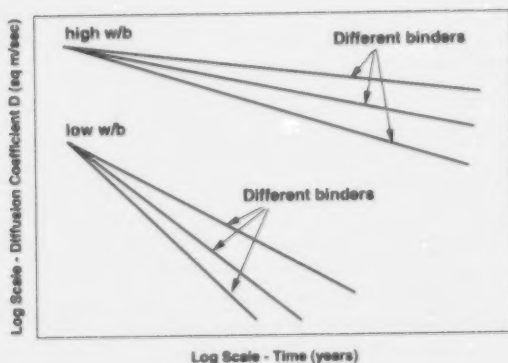


Figure 2: Extrapolation for long-termed D based on double log plots

This correction for D appears to be reasonable. However, the authors intuitively lean towards a limiting value of D . This is based on the consideration of limiting pore refining effect and "maximum" chloride binding effect. The limiting value of D is illustrated in Figure 3. At the moment, the limiting value of D (based on early age values) is guessed rather than rationally predicted. Research work at the CSIRO indicates that D is closely linked to the porosity of the concrete. A possible simple relationship is $D = D_0(1 - e^{-kP})$ where D_0 and k are constants depending on binder type and P is the porosity of the concrete (including cracking). This work is still in progress.

Regardless of what system is used for correcting D , three important points need to be made :

- Correction for variation of D with time is critical in the prediction of chloride penetration into concrete;
- Variation of D with time depends on concrete ingredients and proportions; and
- Variation of D with time depends on environment.

In the previous discussion, the diffusion coefficient D is that obtained from the chloride concentration profile. When D is calculated on each measured point, it has been demonstrated that it is depth dependent and increases with depth. Theoretically, this can be partly explained by the increase in diffusivity with decreased concentration of diffusant solution (Cl^-) at higher depth. The chloride binding capacity has also been observed to decrease with depth [Nagataski et. al 1993; Mangat and Molley 1995]. Boltzmann-Mantano analysis has been used to treat both time and depth dependent characteristics of D [Shewmon 1989; Tumidajski et. al 1995]. The issue of depth dependence of D is still needed to be sorted out in a practical manner. At the moment, it appears that there is no practical need for a correction with a concrete cover of 50 mm or less. For a concrete cover depth greater than 50 mm, a factor of (cover depth/50) should be applied to D for long term prediction of chloride penetration using solutions derived from Fick 2nd law.

It can be summarised that long term prediction of chloride penetration into concrete, which is the most critical module in any service life model, is not a precise process. It is performed with the use of empirical relationships (some with limited application). For practical purposes, it appears that the Fick's 2nd law solutions with suitable corrections can be used to provide a "reasonable" estimation of chloride penetration.

3 Chloride threshold level for steel corrosion and acceptable corrosion rate

Chloride threshold level is usually defined as the concentration below which there is no corrosion of steel reinforcement. For concretes subjected to chloride ingress from an external source, it is difficult if not impossible to determine this value. This is due to the inhomogeneous nature of concrete, cement paste and chloride distribution as noted in the literature [Neville 1995]. Even the definition of "no corrosion" is neither clear nor stated in clear electrochemical terms.

The adaptation of Cl/OH^- ratio derived from testing of steel in alkaline solution [Hausman 1967; Goulda 1970] has been found to be very unreliable when pore solutions are used and different binders are concerned [Cao et. al 1991; Mangat and Molley 1995; Hussain et. al 1996]. Furthermore the pore solution chemistry is expected to change with exposure time due to OH^- movement and the effect of SO_4^{2-} and carbonation. In the laboratory, the authors observed that the "chloride threshold level" varies significantly with binder type, concrete mix design, initial curing and concrete cover. It is also depend on the measurement techniques (eg. halfcell potential, anodic polarization, linear polarization, steel loss, etc.) and adopted definitions.

The sometimes used figures of chloride threshold level of less than about 0.4% by mass of binder or about 0.1% by mass of concrete (total chloride) can be either over optimistic or pessimistic for applications in service life prediction. Due to this additional uncertainty/arbitrary, the authors opted to abandon the concept of chloride threshold level in the prediction of service life. Using laboratory-generated results, the authors observed that when steel corrosion rate is at a reasonable level, the variation of chloride level with respect to binder types, concrete mix design and concrete cover become much less than that observed at low corrosion rate (nearer to chloride threshold level). Similar observation has been given in the literature [Mangat and Molley 1995]. For this reason, it is recommended that acceptable corrosion rate be used in the service life prediction.

It has been suggested that cracking of concrete cover depends on bar size, corrosion rate and cover depth [Weyers 1998; Lopez et. al 1993; Andrade et. al 1993; Schiefl and Raupach 1997]. A corrosion rate of about $2\mu\text{A}\cdot\text{cm}^{-2}$ appears to be a "reasonable" value for acceptable corrosion rate. Available data [Weyers 1998; Lopez et. al 1993; Andrade et. al 1993] suggest that at a corrosion rate of $2\mu\text{A}\cdot\text{cm}^{-2}$, the expected time require to crack a concrete cover greater than 25 mm is greater than 2 years. It would take about 10 years for the crack to reach 0.3mm in width or a 5% section reduction of the bar. Structural performance of the reinforced concrete is not expected to be seriously damaged [Cabrera 1995].

The issue is to determine the chloride concentration at which the steel corrosion rate is $2\mu\text{A}\cdot\text{cm}^{-2}$. Based on electrochemical experimental data generated using a wide range of binders including portland cements and blended cements containing different fly ashes, slag and silica fume at different percentages and water-to-binder ratio, the authors suggest an interim value of 0.2% Cl (total) by weight of concrete. This value appears to be in reasonable agreement with actual site data [Liam et. al 1992]. Work is still in progress to validate this figure and to establish a list of values for different binders and W/B. This chloride concentration is then used in conjunction with the predicted chloride penetration for service life prediction.

4 Prediction of service life of reinforced concrete in marine environment

As mentioned previously, prediction of service life should be viewed as a qualitative or at best semi-quantitative estimation. This is due to the large number of variables involved. The goal in the prediction of service life is to get a reasonable estimation

from which specification requirements, maintenance strategy and other economical decisions can be formulated. Since exactness is not possible and is not the key issue, complicated procedures including lengthy, detailed equations should be avoided. In the authors' opinion, simple procedures are as effective as (if not more effective than) the apparently complicated ones.

The suggested procedure for prediction of service life is that based on Fick's 2nd law with modifications as discussed in the previous sections. For example, if a service life of 50 years is required for a marine RC component with a 50 mm cover, this means that the expected chloride concentration at 50 mm depth after 50 years should be about 0.2% by mass of concrete. Using equation (1) with suggested C_s of 1%, the required chloride diffusion coefficient is about $0.5 \times 10^{-12} \text{ m}^2 \cdot \text{sec}^{-1}$. This is the long term chloride diffusion coefficient. Data are then required to "translate" this requirement back to short term chloride diffusion coefficient needed for specification purpose.

An example of the required data is shown in Figure 3. This figure illustrates the diffusion coefficient data for a 25% fly ash blended cement 50 MPa concrete cured for 7 days. Based on this figure, it is likely that this fly ash concrete will achieve the service life of 50 years. It can be seen from this figure that the measured chloride diffusion for this concrete after 1 and 6 month of immersion in ocean water are about $1.0 \times 10^{-10} \text{ m}^2 \cdot \text{sec}^{-1}$ and $1.9 \times 10^{-12} \text{ m}^2 \cdot \text{sec}^{-1}$ respectively. The D value of $1.0 \times 10^{-11} \text{ m}^2 \cdot \text{sec}^{-1}$ can be used in the specification for fly ash concrete of similar replacement percentage if the test method is based on 28-day immersion. However, this value should not be used when concretes are made with other binder types. This is due to the different response of D vs t .

From this example, the case-by-case feature of service life prediction is illustrated. A 50 MPa/25% fly ash concrete cured for 7 days may satisfy the Australian Standard AS 3600 expectation of 40-60 years service life in marine exposure. Other 50 MPa concrete made with different binders and cured for 7 days may not.

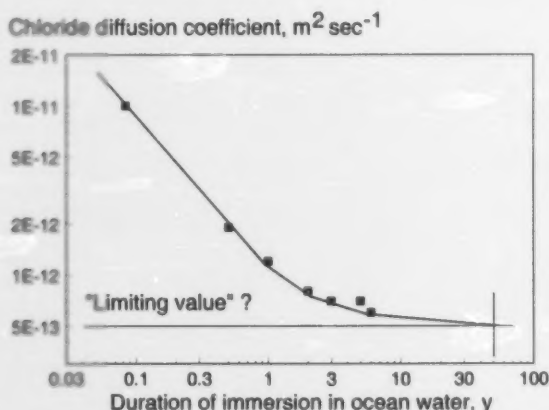


Figure 3: D vs t for a 25% fly ash blended cement concrete, 50 MPa, 7 days cure

5 Concluding remarks

In summary, prediction of service life is a qualitative/semi-quantitative process in which reasonable answers rather than exact answers are sought. This process requires knowledge/experience/data on environment, material behaviour, material/environment interaction and degradation mechanisms. Since service life and routine maintenance are inter-related, predicted service life should be used primarily for the planning of effective inspection/maintenance schedule.

In this paper, a simple procedure for predicting service life of marine RC is proposed with limited scopes. Factors such as cracking/microcracking, temperature, humidity, secondary reactions (eg. sulfate attack, leaching) have not been considered. The key message is that reasonable prediction of service life of marine RC structures is possible as long as data are available and the limitations are known.

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PART ONE

**DURABILITY OF CONCRETE
MATERIALS AND
STRUCTURES**

**STUDIES ON STRENGTH AND
DURABILITY OF CONCRETE**



10 DURABILITY OF AUTOCLAVED AERATED CONCRETE

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Abstract

Field inspections and further tests have been carried out in order to identify damages and degradation factors for the external wall components made of autoclaved aerated concrete. Within this project 24 industrial buildings in the area of Örebro and Kumla in Sweden have been inspected to get information about all damage types and to rate the status of the AAC-plaster system. The buildings had one of three different types of surface treatments. They were built over three decades beginning in the 1960's. The inspected external AAC-walls were either untreated or if treated, organically or inorganically based thin plaster systems were applied. In-situ measurements of the moisture content within the first 15 mm below the surface were carried out by a conductance type electric moisture meter.

The evaluation of damages was performed with regard to differences in plaster system, plaster thickness and colour, AAC's density, location and orientation of the walls.

To detect possible changes in properties of the AAC, capillary water absorption tests and water vapour diffusion tests were performed at core samples from three buildings of different age and surface treatment.

Of main interest have been moisture-related damages in organic and inorganic based plaster systems. The level of moisture content is directly related to damage and is also connected to the degree of damage. AAC-walls treated with organic plaster systems showed most moisture-related damage. Where organic plaster systems were used, the level of degradation of the AAC was in general conspicuously higher than that at the walls which were coated with inorganic plaster systems.

Damage causes and possible degradation systems are provided for the different plaster systems.

Keywords: autoclaved aerated concrete, durability, plaster

1 Introduction

Autoclaved Aerated Concrete (AAC) is a building material, which is used all over the world. It is a purely mineral-based building material made from sand, water, limestone and a very small amount of aluminium powder. These raw materials are combined to provide a building material with a large number of air pores. One way to use the building material is to construct external walls of industrial buildings with AAC-components. In Sweden, thin plaster systems are usually applied to the surface of these walls.

Since little is known about the degradation processes of AAC and the interaction of AAC and the plaster system under long time exposure to the environment, the aim of the work which underlies this paper, is to detect and to describe all damage types which occur at the buildings, at the components and in the materials during their service life. One further scope is to identify degradation factors and mechanisms of plastered AAC-wall systems. It is desirable to develop a methodology for assessing the progressive degradation effects of local agents and other circumstances on components and assemblies to identify the future maintenance and replacement needs of a building.

2 Moisture and properties

Autoclaved aerated concrete is a porous building material. As a consequence, there is a strong interaction between water, water vapour and the porous system. Most properties of porous materials depend on the moisture content (Künzel 1971; Svanholm 1983; CEB-Manual 1978; Yxhult AB 1993).

- The compressive and tensile strength vary with the moisture content. There is a distinct increase in strength after drying out from a moisture content of 10% to almost 0%.
- The modulus of elasticity in compressions shows an increase, in drying, of about 15%.
- The thermal conductivity increases linearly with the moisture content.
- The permeability varies according to the moisture content in the material – the lower the moisture content in the pores the greater the permeability.
- Shrinkage and creep increase with excessive drying.
- Frost damage can appear if the moisture content of AAC exceeds a critical value.
- The resistance against chemical attack depends on the moisture content.
- The resistance against accidental fire also depends on the amount of moisture in the material, and is higher with higher moisture content.

Under the given climatic conditions, the moisture content is determined by the total porosity and the pore size distribution. The bulk density of steam-cured aerated concrete is within the range of 300 to 1000 kg/m³. The most common dry densities for load-bearing products and reinforced AAC-elements are 400 kg/m³ and 500 kg/m³. These values refer to the oven-dried material. The moisture content of the delivered material may be 20 to 35% by weight. This moisture dries out gradually until an equilibrium is reached at 4 to 6% by weight in external constructions (CEB-Manual 1978). The drying out process lasts about one to three years under normal

conditions.

The structure of autoclaved aerated concrete is micro-crystalline and formed mainly by tobermorite platelets, which are more stable than normal cured concrete (RILEM Recommended Practise). The structure is characterised by pores and can be clearly divided into three regions. One of these regions consists of air pores with a radius of 50 to 500 μm introduced by gas development or surface-active agents during the manufacturing process. Another region is identified by microcapillaries of 50 nm or less, which is the gap of the hydration products developed in the wall between the air pores. In the third region, there are a few pores of 50 nm to 50 μm , which are called macrocapillaries (Tada and Nakano 1983).

There are different moisture transport mechanisms in porous materials. In a dry state, all pores are empty and therefore water vapour diffusion dominates. If the moisture content increases a little, the AAC enters a state where adsorption of moisture to the capillary wall takes place and capillary condensation occurs in the fine capillaries. This condition is hygroscopic, and moisture movement is dominated by water vapour transport. With a further increase in the moisture content, the microcapillaries are completely filled with water. This condition occurs over a long period of time with just diffusion of water vapour, but is reached very rapidly when in contact with free water. If the moisture content becomes very high, some pores are filled by capillary condensed water. In this case, these fine pores cannot contribute to vapour diffusion any more and water driven by capillary forces migrates through them instead.

Excessive drying below the equilibrium results in an increase in shrinkage and even in shrinkage cracking. Shrinkage increases if the AAC is alternately humidified and dried (Duorjadjkin and Malinowski 1965).

3 Conditions of field investigations

During the field inspection, 24 industrial buildings were inspected in December 1997 and February 1998. The buildings were predominantly industrial halls commonly used as mechanical workshops or stores. The oldest building was built in 1960 and the youngest in 1996. The range of age is therefore 36 years.

All buildings were situated in local industrial parks in Örebro and Kumla in Sweden. The global climate in that region can be assumed as being equal for all buildings. It is moderate and is characterised as an inland climate with moderate SO_2 levels ($< 20 \text{ mg/m}^3$) (Nevander and Elmarsson 1994).

The external walls of all inspected buildings were constructed with components of autoclaved aerated concrete (AAC) manufactured in Sweden by Yxhult AB. The wall elements were rectangular and were fitted with groove and tongue joints.

The surface treatment of the external walls of most buildings was with thin plaster systems. The investigation distinguishes only between mineral and organic plasters. The organic plasters for AAC are normally based on an acrylic resin binder with siliceous fillers and grains forming the surface texture. The inorganic thin plaster systems usually consist of a two-layer lime-cement mortar. The thickness of the plasters varies from 0.5 to 5 millimetres. The walls of some inspected buildings were only thinly painted or were untreated.

4 Damage and causes

The investigation of the 24 industrial buildings showed a multitude of varying damages. Many types of damage occurred frequently and mostly with similar causes.

4.1 Construction damage

Construction damage is damage that is actually caused by false design having an impact on structural elements of the building.

4.1.1 Cracks between structural components

All constructions consist of several different structural components. They have different functions and therefore their constitution and behaviour varies. The connection of these components often causes problems, and cracks appear at the joints. These cracks are usually in the plaster if the connection allows a certain movement of entire adjacent components (for example the connection between normal concrete and AAC). The reasons for the movements are usually the component's different behaviour with regard to thermal expansion, creep and shrinkage.

4.1.2 Cracks within AAC-components

Stresses within a structural component can have several causes. If stresses exceed the ultimate strength of AAC, they cause cracks. Sometimes deep cracks arise due to poor performance of the support for the AAC-elements. Connecting the AAC-elements with joint sealers, which are too stiff, causes cracks. Deterioration by ageing causes stiffness within the joint sealers, which does not allow the elements to move properly.

4.2 Mechanical damage

The damage found most often was mechanical damage. This can occur at any place and in any size in a wall. Scratches and impacts of all shapes flaw the surface treatment, and the AAC. The reinforcement is often uncovered or even damaged. Traffic consisting of cars and other vehicles is the main cause of damage. Therefore mechanical damage appears particularly at places with a high traffic volume (e.g. ramps), and at the lower parts of the wall, usually up to a height of three meters above ground.

4.3 Damage caused by bad design

All bad-design damage in this study was actually caused by water or moisture problems. The degree of damage and the appearance vary quite strongly. Figure 1 illustrates where these types of damage usually appear. Damage connected to poor performance of the concrete base occurs extremely often. If the base is not high enough, rainwater splashes from the ground to the wall and imbrues the surface of the wall up to a height of about 0,8 m. The moisture content within the AAC can then become very high. This promotes the occurrence of cracks due to shrinkage, and the growth of algae and lichens, as well as strong discolourations. The degree of damage depends on the direction of the wall, the microclimate and the type of ground in front of the wall. Asphalt or any kind of plain, hard surface (e.g. slabs) intensifies the

effect of humidification.

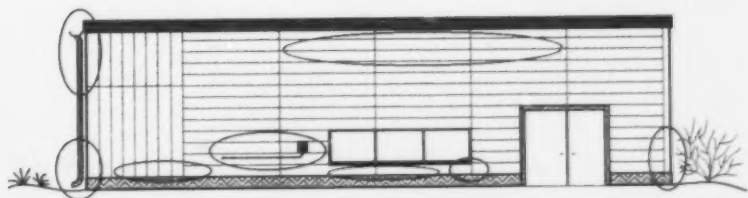


Fig. 1: Moisture-related damage at a typical industrial building. Common locations of damage are indicated

Where AAC is not covered with plaster, water infiltrates and diffuses behind the properly plastered surface. Then, there is little chance for it to dry out if the plaster is tight. A wall can have areas where the moisture content at its surface is very high. If the moisture content exceeds an assigned value and the surface is not toxic, algae and lichens can grow on the plaster or the AAC and strong discolourations can occur. This phenomena occurs quite frequently. It is evidence of a high moisture content of the wall wherever it occurs.

The performance and design of the roof edge have a high influence on the appearance of damage. A common mistake is to fix any equipment onto the surface of the wall, for example cables or boxes, so that rainwater can drop down on the facade, splash against it, or sometimes it cannot be drained off. All problems combined with high moisture contents of AAC can occur as discolourations and algal growth. Deep and wide cracks in irregular patterns occur, which introduce the peeling of plaster.

4.4 Frost damage

Frost damage was rare and was usually caused by design errors, which made it possible for water to penetrate the AAC regularly. Where the AAC was laid open and was not, or was inadequately plastered, frost damage appeared if the close surrounding of these places was coated with a water vapour tight plaster.

4.5 Cracks and crack pattern

Cracks can have different causes. They occur both as vertical and horizontal cracks as well as without a specified pattern. The causes are often moisture-related, as for example shrinkage cracks, or cracks which follow the reinforcement and the production joints of the AAC-elements. These inhomogeneities are weak points considering AAC's behaviour under the influence of moisture and temperature variations.

4.6 Damage due to bad use and lack of maintenance

Regular maintenance is necessary if the plaster tends to age quickly. Great problems occur if the plaster gets too old and has already lost its function. Protection against water ingress is not present and the susceptibility of the plaster-AAC system to damage increases. Missing or badly performed repairs of damage can aggravate the condition of the walls.

4.7 Damage and degradation of plaster

The surface of the wall is affected by weather and by the whole environment of the building. It can show discolourations and bleaching caused by solar radiation, disintegration, weathering, dust deposit and traffic. Degradation of plaster occurs as peeling, bleaching, chalking or the formation of blisters. Often the bond between the plaster and the substructure is lost, where the expansion of the coating increases due to thermal expansion and changes in the moisture content.

5 Evaluation of field inspections

The field inspections showed a multitude of different damage types and provided values of the walls' moisture content. A gradation of the damages was performed with regard to the different damage types, the degree of damage and the orientation of the walls to all points of the compass.

During the field inspections the moisture content below the outer surface of the external walls was measured by a conductance type electric moisture meter equipped with non-insulated pin-electrodes of 15 mm. The instrument provides the value of the highest moisture content between 3 and 15 mm within the AAC below the surface. The measurements were executed at different heights above the ground. In general the moisture content in the first AAC-element above the concrete base was higher. Average values of the moisture content over all of the walls were 24% by weight for the first AAC-element and 20% by weight for the second AAC-element.

A grading of the measurement was done with regard to the location of the measurements. The evaluation distinguishes between measurements next to the corners and at central areas of the wall. Measurements in unprotected damage spots (where the plaster was removed for example due to mechanical damage) provided information about the permeability of the plaster system. If the moisture content in a spot was similar to the moisture content at a place next to it, which was well plastered, it could be stated that the plaster system was very permeable to water vapour. This was seen mostly with mineral plaster systems.

5.1 Results of field inspections

It can be stated that the frequency of damage is much higher if the moisture content of a wall is high. Damage that can be related to a high moisture content in the AAC, is for example, moisture-related cracks, almost all design damage as growth of algae and lichens, discolourations and frost damage. Furthermore damage occurs due to inhomogeneity of AAC and peeling of plaster more frequently, where the moisture level is high within the AAC. Algal growth and moisture-related discolourations occurred more frequently at the northern and western facades of the buildings and

were worst at base elements close to the ground. Most frequent types of cracks seen during the inspection, were vertical cracks at the southern side of the buildings. They usually appeared in base elements with a small crack width (0,1 mm – 0,4 mm). This can be explained by a higher amplitude of temperature and moisture content due to solar radiation, which causes an increased tendency to shrinkage. Frost damage occurred in only a few cases. They were usually caused by design errors. Damage that appeared most frequently, was mechanical damage. It was normally concentrated in the first two metres above ground and occurred most frequently next to doorways, portals and entrances or where the ground in front of the wall allowed vehicles to come too close.

5.1.1 Plaster type

At buildings where degradation of the organic plaster system was already noticeably active, the moisture content below the outer surface was in general conspicuously higher than in mineral plaster systems. The measurements at walls with organic plasters provided an average value for the moisture content of 24% by weight. Mineral plaster systems provided an average moisture content of 19% by weight. The difference in moisture content is confirmed by the results of the inspection. Organic, acrylic based plasters showed a multitude of different damage types and most of them were cracks and algal growth obviously due to a high moisture content in the plaster and AAC. Only a few signs of degradation were observed in the inspections of the inorganic plaster systems.

5.1.2 Plaster thickness

The frequency of damage and the level of degradation decreases with the thickness of the plaster. The walls, which were protected by plasters with a thickness of at least 2 mm, showed the best conditions.

5.1.3 Colour of plaster

To examine the influence of the colour of the surface treatment, the facades of all 24 buildings were graded into dark and light colours. In general, the condition of light coloured facades was much worse. They showed more moisture-related damage and the crack frequency was higher. The in-situ measurements provided an average moisture content of 24% by weight at the light facades. This is noticeably higher than the measured moisture content of 17% by weight at the dark coloured facades. Thus, it can be assumed that the moisture level within the outer parts of a wall's cross-section is related to the colour of the surface treatment. Dark facades dry out more quickly because the temperature rises faster and to a higher level, due to solar radiation, after penetration by rain.

5.1.4 Density of AAC-components

A higher density of the AAC has a good influence on the condition of the wall. All moisture-related cracks occurred more frequently in AAC with a lower density. The results of the moisture measurements support this assumption. The average moisture content under the surface of walls with a density lower than 500 kg/m³ was 25% by weight. AAC-walls of a higher density had an average moisture content during the measurements, of 19% by weight. The higher moisture content in AAC-

walls of lower density is caused by an increased tendency of capillary water absorption of the material.

6 Capillary suction and water vapour diffusion

Capillary suction tests and water vapour diffusion tests were executed at core samples that were taken from external walls of three buildings. The tests were intended to help detect possible changes in properties of the AAC due to degradation in the course of time. The buildings were of a different age and had different surface treatments (untreated, organic and mineral plaster systems). The tests were executed at different depths of the core samples, which comprised the entire cross-section of the external walls. The tested sections were the outer surface of the wall's cross-section, ten millimetres below the outer surface, the centre of the core sample and the inner surface.

6.1 Results of the capillary suction test

The test showed that the amount of absorbed water is significantly higher in sections of the organically plastered wall. A possible answer for this behaviour is that the plaster and the AAC have got little micro-cracks that support the water absorption. Sandin showed the negative effect of cracks on the moisture balance of organic plaster systems that have been applied to AAC (Sandin 1983). Cracks can effect an increased water uptake. Increase in movement from wetting and drying out procedures, which will secondarily cause more severe cracks is a possible after-effect. After several years, organic plasters with acrylic binders are strongly affected by ageing and degradation. They lose their elasticity and plastic deformation capacity. They are no longer able to achieve all deformations of the AAC. Figure 2 shows the amount of absorbed water at four surfaces within the cross-section of the organically plastered external wall. The plastered outer surface absorbed most, the central surface least water.

Although the organically plastered building is 22 years younger than the other two tested buildings, it showed most damage. It furthermore exhibits the highest values for moisture content measured with the electrical method.

There was almost no difference in the amount of absorbed water between the outer surface of the untreated and the mineral-based plastered wall. It is obvious that the mineral plaster is very permeable and can only have a very small influence on capillary water absorption of the surface.

The central section of the core samples absorbed the least amount of water. It can again be interpreted that the central section is not as affected by the environment as are the surfaces. Carbonation may be the reason for the outer and inner sections and the section 10 mm below the outer surface to absorb considerably more water than the central section. Carbonation is a process where carbon dioxide in the air and water reacts with the calcium hydroxide in the concrete. This reaction can cause a change in AAC's structure. These inferences are assumptions and can be verified only by further chemical and microscopic examinations of the structure.

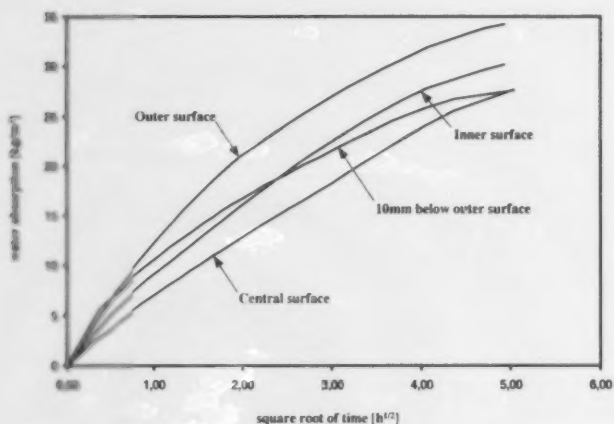


Fig. 2: Amount of absorbed water in an external AAC-wall with organic plaster

6.2 Results of the water vapour diffusion test

To determine the diffusion factor at different depths of the wall cross-section, the 'Wet Cup-Method' was used with a relative humidity difference of 66 to 100% at 20° Celsius. The test showed that the outer surface of the organically plastered wall has a very small diffusion coefficient, whereas below the outer surface, the diffusion coefficient is highest. The AAC below the outer surface would guarantee that the wall dries out very quickly. However, this is hindered by the organic plaster system with its high diffusion resistance factor. The capillary water absorption test illustrated that the amount of absorbed water at the outer surface and 10 mm below the outer surface was highest at the organically plastered wall, which is probably caused by a change in the AAC's structure and by micro-cracks. The system contributes to an accumulated moisture content. The duration of a moist state within the AAC is therefore longer than in inorganic or untreated systems. This surely leads to an accelerated degradation process of the AAC.

The test of the untreated wall showed that the diffusion coefficient of the untreated outer surface was considerably higher than that below that surface and the centre of the wall cross-section. This guarantees a faster drying out process after penetration. It seems to have a good effect on the wall since the surface of the untreated facade showed nearly no cracks or moisture-related damage during the inspection. The higher value for the diffusion coefficient indicates that the structure of the AAC at the surface seems to be influenced by the environment.

The values for the diffusion coefficient of the minerally plastered AAC guarantee a fast drying out of the penetrated wall and differ not very much along the cross-section. This provides for the assumption that a permeable mineral plaster does not have a bad effect on the structure of AAC.

7 Conclusions

Based on these field studies and the laboratory tests it can be concluded that inorganic plaster systems generally degrade to a lower degree than organic systems. The users of the buildings do not usually consider the different maintenance requirements of the different plaster systems. Omission of maintenance of organic thin plaster systems is the start of degradation and results in further damage. The time for renewing treatments of organic plasters is much shorter than for inorganic plaster systems. Organic, acrylic based plasters showed a multitude of different damage types and most of them were cracks and algal growth due to a high moisture content in the plaster and AAC.

Only very few signs of degradation were observed during the inspections of the inorganic plaster systems. Thus, mineral surface treatments seem to be more suited for a moderate climate. Inorganic plasters can allow higher and quicker capillary water absorption, but also a shorter drying out period due to their high water vapour permeability. Therefore, they stay dryer in the outer part of a wall between periods of saturation although they could have higher peak values of moisture content.

It can be stated that the occurrence of damage is much higher if the moisture content of a wall is high. Visible moisture-related damage on a facade appear for example as cracks, frost damage, growth of algae and lichens, discolourations and peeling of plaster. Although the field inspections were executed in a region with a moderate climate, the visible effects of a more severe microclimate were found to dominate in all damaged areas seen during the inspection. Design factors combined with the microclimate in certain points of the building, can effect the plaster and the AAC more severely than the influences of the average meso and global climate. The detailed design of buildings, for example, the height of the concrete base and the performance of the facades, are very important factors for durability.

8 Acknowledgement

Kjell Nygren, M.Sc., at the Swedish manufacturer of AAC, Yxhult AB, has given highly appreciated support in the structuring of the work and also in the discussion and analysis of the results.

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11 EFFECT OF MATRIX PROPORTIONS AND CURING TEMPERATURE ON THE PHYSICAL, MECHANICAL AND DURABILITY PROPERTIES OF LIME-NATURAL POZZOLAN PASTE

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Abstract

This paper deals with the effect of curing temperature and matrix proportions on the properties of lime-natural pozzolan pastes. The results indicate that the 70°C curing gives the maximum compressive strength for pastes with a lime/pozzolan ratio of 35% and water content 45%. Curing at elevated temperature has a great accelerating effect on the pastes. The increase in water content causes a reduction in mechanical strengths. The setting time of pastes increases with an increase in lime and water content, ranging from 37 to 70 hours. X-ray diffraction analysis indicates that calcium silicate (Ca_2SiO_3) and gypsum ($\text{CaSO}_4 \cdot 2\text{H}_2\text{O}$) are the hydration products in the lime-pozzolan pastes. There is no considerable difference between the volcanic tuff and paste in the case of pore structure. The most important change in the paste takes place in the fraction of the larger pores (larger than 0.1 μm of radius), that shows a significant reduction. The 70°C cured pastes with lime/pozzolan ratios of 35% and 40% and water content 45% exhibit excellent frost resistance. The results of statistical analyses show a clear correlation between matrix proportions, curing temperature and mechanical strengths. The proposed quadratic equations are found to be an accurate representation of these relations.

Keywords: Hydration products, lime-pozzolan paste, mechanical strengths, pore structure, frost resistance.

1 Introduction

Pozzolans are natural or artificial materials that contain silica and alumina. These materials have been used to obtain hydraulic binders for over a thousand years. The first natural pozzolans to be used in building construction were the volcanic deposits. These materials, when finely ground and mixed with lime, sand and water

yielded mortars of superior strength, which also resisted deterioration in fresh or salt water (Malinowski and Garfinkel 1991; ACI Committee 232 1994; Massazza 1993; Larew 1976). The use of lime-pozzolan mortars for cementitious purposes declined gradually because of their relatively slow setting and hardening characteristics, after the invention of portland cement. But compared with the production and use of portland cement, these materials contribute to cost and energy savings. For these reasons, lime-pozzolan mortars are still commonly used in some developing countries (Coad 1974). However, the production of this type of building material is a time consuming process, since the rate of strength development of lime-pozzolan mortars cured at room temperature is very low. The solution to the problem of increasing the early strength of lime-pozzolan mortar was obtained by accelerating the pozzolanic reaction between pozzolan and lime. In a study, (Takemoto and Uchikawa 1980), it was reported that the reaction rate of pozzolanic materials is proportional to the specific surface area squared. Increasing fineness of pozzolan increased the early strength of mortar (Dhir et al. 1986; Giergiczny and Werynska 1989; Watt and Thorne 1966). Recent studies have indicated that the pozzolanic reactions can be accelerated by temperature (Lea 1974; Day 1992; Shi and Day 1993a) and by the presence of chemical admixtures, such as alkalis and sulphates (Shi and Day 1993b, 1994; Shi et al 1994). It has been found that the strength of lime-pozzolan mortars increases as the curing temperature rises, but different pozzolans show different responses to temperature rise (Alexandar and Wardlaw 1955).

In this study, the effect of matrix proportions (lime/pozzolan, water/lime+pozzolan), and curing temperature on the physical, mechanical and durability properties of lime-natural pozzolan pastes is investigated. The microstructure of pastes is also examined. A statistical correlation is found between matrix proportions, curing temperature and mechanical strengths.

2 Materials and method

Volcanic tuff in the area around Konya was used in the production of lime-pozzolan pastes. Its chemical composition is given in table 1. Volcanic tuff consists of glassy particles of variable silica (SiO_2), alumina (Al_2O_3) and iron oxide (Fe_2O_3) content. The most abundant phase is silica. The $\text{Al}_2\text{O}_3 + \text{Fe}_2\text{O}_3 + \text{SiO}_2$ proportion affecting the pozzolanic activity within the material is 81.6 % that goes beyond the limit of 70 % determined in TS 25 (Trass). Thus, the material differs between neutral and acid rocks and is classified in the group of dacite. The pozzolan was ground in a small ball mill to certain fineness. Its specific surface area is $4875 \text{ cm}^2/\text{g}$. The density of volcanic tuff is 1.26 g/cm^3 . The proportion of bulk water absorption in volcanic tuffs ranges between 30.4- 44.75 %. The capillary coefficient in the volcanic tuffs varies from $0.0597 - 0.0678 \text{ cm}/\sqrt{\text{min}}$.

Table 1: Chemical compositions of volcanic tuff and lime

Material	Chemical comp. (%)									
	SiO ₂	Fe ₂ O ₃	Al ₂ O ₃	CaO	MgO	Na ₂ O	TiO ₂	K ₂ O	SO ₄	LOI
Vol.tuff	68.4	11.15	2.05	0.35	0.03	0.28	0.37	2.2	4.58	10.5
Lime	-	-	2.84	92	4.01	-	-	-	-	-

A commercial hydrated high calcium lime was used in the pozzolanic reactivity test. The lime composition used in this investigation is also given in Table 1. The density of lime is 2.25 g/cm³. The pozzolanic activity test appropriate to TS 25 with SiO₂ sand arranged according to the granulometry of TS 819 (Standard sand for RILEM-CENBUREAU) was made. The valency for compressive strength after 7 days was found as a minimum 20.34 MPa, as a maximum it was 29.5 MPa. Whereas, minimum value recommended in the TS 25 is 4 MPa for compressive strength. The lime in amounts of 30 to 40% of the weight of the pozzolan was used. For each lime content, the water/(Lime+pozzolan) ratio was changed from 45 to 55%. These parameters were determined from an extensive series of optimisation tests. Mixing and sample preparation were performed at room temperature. The samples were produced in 4x4x16 cm steel moulds. In the production of samples, a plastic bag (polyethylene) was used for the nonevaporation of water vapour according to TS 25. The heat of etuve was accepted 20°C at the beginning and the heat was risen gradually 20°C per hour. Specimens in the plastic bag were stored under 90 % relative humidity (RH), at 20±2°C for a period of 1 day and then cured in different etuve having temperatures of 50,60,70±2°C for six days. Three samples were taken out four hours before the testing time to cool. The samples were tested in compression and in flexure after 7 days. The strength results given are an average of the three specimens. The average coefficient of variation of these results is less than 10 %. The samples were also tested for porosity, setting time, capillary coefficient, freeze/thaw and mineralogical composition and microstructure characteristics. Freeze/thaw tests were conducted according to TS 699 (Methods of Testing for Natural Building Stones). A cycle consists of thawing the sample for 4 hours in water at 20°C followed by 4 hours freezing at -20°C.

In the mineralogical analysis, the pastes were cast into the cylinder mould (25 mm diameter and 50 mm high) and cured as previously mentioned. The cylinder was cut into small fragments and then dried in an oven at 105°C. The dried fragments were ground and tested using a Philips X-Ray Diffractometer. Hydration products were identified with the help of the characteristics of peak intensity. The pore structure of lime-pozzolan pastes was analysed by using mercury intrusion technique. The mercury porosimeter used was a Micromeritics Autopore II 9220. Samples with a weight between 0.758-1.137 g were used, making two tests for each sample.

3 Results and discussion

3.1 Setting time

Setting times of the lime-natural pozzolan pastes were determined by the vicat needle. The tests were carried out at 20°C temperature and 90% relative humidity. The effect of matrix proportions on the setting time of pastes is shown in Table 2. In general, the pastes exhibit a long setting time. This may be attributed to the slow pozzolanic reaction between volcanic tuff and lime. The increase in water and lime content increases the setting time from approximately 37 to 70 hours. At 35% lime/pozzolan ratio, there is no significant increase in the setting time that ranges from 37-39 hours. However, considerable retardation in setting time can be expected for these pastes with a water content higher than 50 %.

Table 2: Setting times and mechanical strengths of lime-pozzolana pastes

Matrix	Ratio	Setting Time (hours)	Curing temperature (°C)					
			50		60		70	
Lime Pozz. (%)	Water content (%)		Compressive strength (MPa)	Flexural strength (MPa)	Compressive strength (MPa)	Flexural strength (MPa)	Compressive strength (MPa)	Flexural strength (MPa)
30	45	37	20.45	1.67	23.72	1.78	24.55	2.75
	50	43	18.37	1.29	21.65	1.58	22.87	1.86
	55	62	15.43	0.98	18.45	1.35	19.42	1.47
35	45	39	22.87	1.83	27.32	1.97	30.5	2.98
	50	52	18.53	1.37	22.47	1.41	25.11	1.95
	55	65	14.24	1.09	17.48	1.18	19.46	1.54
40	45	46	20.42	1.55	24.86	1.32	25.64	1.89
	50	57	17.51	1.01	21.54	1.12	22.72	1.21
	55	70	15.65	0.92	17.32	0.97	17.79	3.05

3.2 The effect of curing temperature and matrix proportions on the strength of lime-pozzolan pastes

The strength of pastes cured at 50,60 and 70°C is given in Table 2. In general, the load-bearing capacity of pastes increases slightly with the curing temperature between 50 and 70°C. Curing at 60°C increases the 7 days compressive strengths of the pastes by 15-22% relative to curing at 50°C. However, the marked effect of the curing temperature on the mechanical strengths of the pastes is recognised at 70°C. Curing at this temperature gives a maximum compressive strength (30.5 MPa) for the pastes with a lime/pozzolan ratio of 35% and water content 45%. From the result in Table 2, it is found that matrix ratios affect and modify the mechanical strengths of the pastes. The increase in water content of the paste (having a lime/pozzolan ratio of 30%) from 45% to 55% causes a reduction in mechanical behaviour, the magnitude of the decline being largely dependent upon the quantity of water in the mixture. This trend is also valid for the pastes with lime/pozzolan ratios of 35 and 40%. However, the effect of the lime content on the mechanical strength of the pastes is quite different. The compressive strength of the pastes reaches the maximum at a lime content of 35% and then tends to decrease slightly with increasing lime content.

The effect of curing temperature on early strength is also assessed. The 20°C curing temperature is insufficient to produce 1day strength in the pastes, but at

120°C, 10 hours later, the strength of pastes is about equal to the paste cured at 70 °C for 6 days. As a result, curing at elevated temperature has a great accelerating effect on the lime-pozzolan pastes.

Figure 1 shows the mechanical strengths of ground pozzolan autoclaved with various proportions of hydrated lime at 120°C for 5 and 10 hours. The pozzolan fully reacts with 35% lime after at least 10 hours at 120°C like fly ash, quartz and other siliceous materials that consume large quantities of lime. The optimum amount of lime activator is found to be 35%, resulting in high strengths. When higher contents of lime are added, strengths decrease.

The relationship between matrix proportions of the pastes, curing temperatures and flexural strength is given in table 2. The data show a similar type of behaviour to that observed in the case of compressive strength with one exception. The sample with 40% lime content, 55% water content, cured at 70°C shows the highest flexural strength of all samples when one would expect that the high water content would result a lower flexural strength. This is also supported by the capillary coefficient results in Table 6. This apparent contradiction is somewhat difficult to explain at the present time, but can probably be attributed either to a particular structure formed during curing or more simply, to experimental mistakes.

In an attempt to formulate the statistical relationships among mechanical behaviour, matrix proportions and curing temperature, data was analysed to generate predictive equations. A statistical analysis system was used to develop regression equations. Figures 2 through 4 present the results of statistical analyses where the mechanical behaviour of lime-pozzolan mortars is plotted against curing temperature and matrix proportions respectively. The proposed equations with their statistical relevance are also shown. The most suitable regression equations are found to be of quadratic form. The correlation coefficients (r^2) indicate that there is a significant correlation between mechanical behaviour, matrix proportions and curing temperature. As a result, both matrix ratios and curing temperature can be properly utilised to describe the mechanical behaviour of lime-pozzolan paste

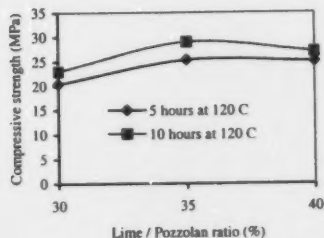


Fig. 1: Relationship between compressive strength and percentage of lime added to ground pozzolan for various periods of autoclaving at 120 °C

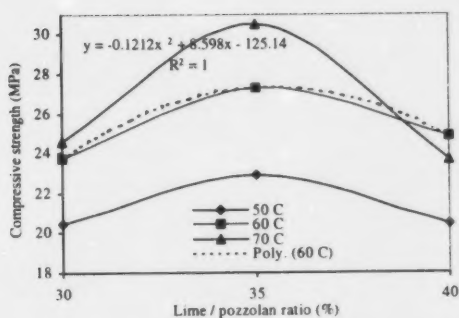


Fig. 2: Effect of lime content on strength of pastes with 45 % water content

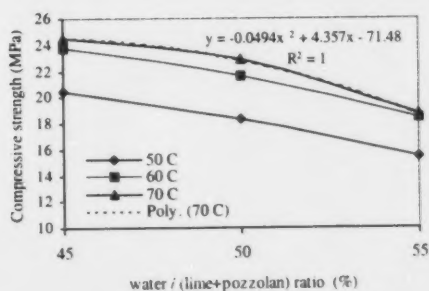


Fig. 3: Effect of water content on strength of pastes with 30 % lime content

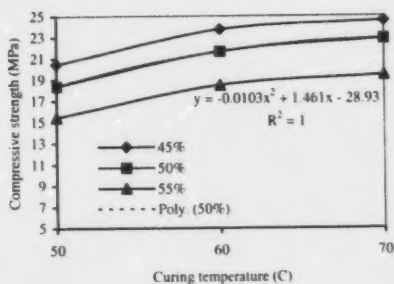


Fig. 4: Effect of curing temperature on strength of pastes with 30 % lime content

3.3 XRD analysis

In the mineralogical analysis made with the X-Ray Diffractometer, a high proportion of jarosite, a low proportion of quartz, opal cristobalit and opal amorph were found in the volcanic tuff (Table 4.) Even if there is a composition in the lime-pozzolan paste similar to the volcanic tuff, new formations resulting from a chemical reaction take place. In the lime-pozzolan paste, proportions of jarosite, quartz, opal cristobalit, and opal amorph change and turn into calcium silicate and gypsum.

3.4 Pore structure

The pore size distribution and other parameters related to the pore system of the pastes and volcanic tuff were obtained by using mercury intrusion technique. The tests were carried out on the paste with a lime/pozzolan ratio of 35% and water content 45%, and the volcanic tuff. Figure 5 shows the pore size distribution curves.

Table 4: The results of XRD analysis

Volcanic tuff	Lime-pozzolan paste
Jarosit % 60-70 $\text{KFe}_3(\text{SO}_4)_2(\text{OH})_6$	Jarosit (low proportion)
Opal cristobalite % 15-20 SiO_2	Opal cristobalite (low proportion)
Quartz % 5-10 SiO_2	Quartz (same proportion)
Opal amorph % 10-20 $\text{SiO}_2 \cdot n\text{H}_2\text{O}$	Opal amorph (low proportion)
	Gypsum $\text{CaSO}_4 \cdot 2\text{H}_2\text{O}$ (hydration product)
	Calcium silicate Ca_2SiO_5 (hydration product)

The volcanic tuff has a considerable quantity (70 %) of pores with size larger than $0.02\mu\text{m}$. More than 50% of the available pore space consists of pores with radius between $0.02\mu\text{m}$ and $6\mu\text{m}$. The volcanic tuff contains only about 8.5 % large pores ($> 6\mu\text{m}$). There are pores of an approximate diameter of $0.01\mu\text{m}$ throughout this type of material. There is no considerable difference when comparing the results obtained for the volcanic tuff and lime-pozzolan paste either in the case of the porosity, total intrusion volume or average diameter of pores (Table 5). The most important change in the paste takes place in the fraction of the larger pores (larger than $0.1\mu\text{m}$ of radius), that shows a significant reduction. The porosities and capillary coefficients of lime-pozzolan paste are given in Table 6. The minimum porosity of the paste is determined to be about 43.5 %. Increasing the lime content at constant ratio of water/lime+pozzolan does not change the porosity significantly as measured by water absorption. For pastes in which lime content is kept constant and the water content is increased the porosity is increased. On the other hand the porosity of the pastes is only slightly affected by the variation of the curing temperature, but tends to be linearly increasing with increasing curing temperature. Capillary coefficient results listed in Table 6 indicate a similar trend to that observed in the case of porosity. In general, the capillary coefficient of pastes increases with an increase in curing temperature. However, a different trend of capillary coefficients is observed for the pastes with a lime/pozzolan ratio of 30%. The capillary coefficient of these pastes attains to the maximum at a curing temperature of 60°C , and then decreases remarkably with rising curing temperature.

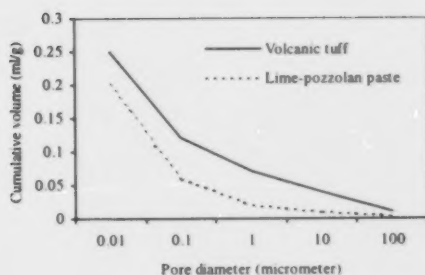


Fig. 5: Pore size distribution of paste and volcanic tuff

Table 5: Pore system values

Material	Total intrusion volume (ml / g)	Average pore diameter (μm)	Porosity (%)
Volcanic tuff	0.3354	0.011	43.37
Lime-pozz. paste	0.2965	0.009	39.7

Table 6: Porosities and capillary coefficients of lime-pozzolan pastes

Matrix	ratios	Curing Temperature ($^{\circ}\text{C}$)					
		50		60		70	
Lime Pozz. (%)	Water content (%)	Porosity (%)	Capillary coefficients $\text{cm} / \sqrt{\text{min.}}$	Porosity (%)	Capillary coefficients $\text{cm} / \sqrt{\text{min.}}$	Porosity (%)	Capillary coefficients $\text{cm} / \sqrt{\text{min.}}$
30	45	44.7	0.0325	45.5	0.0385	46.8	0.0155
	50	50.6	0.0414	51.7	0.0495	53.4	0.0273
	55	54.8	0.0487	55.9	0.0543	57.5	0.0308
35	45	43.5	0.0324	44.8	0.0379	45.5	0.0266
	50	51.6	0.0445	52.7	0.0456	53.8	0.0577
	55	56.9	0.0549	57.4	0.0524	58.1	0.0692
40	45	45.3	0.0367	46.2	0.0463	47.4	0.0626
	50	51.5	0.0532	52.6	0.0748	53.8	0.0893
	55	54.3	0.0645	55.8	0.0985	56.7	0.0756

3.5 Durability

The durability of lime-pozzolan pastes was examined by determining its frost resistance. The test procedure consists of freezing in air and thawing in water. The samples were examined after about every 5 freeze-thaw cycles. The scaled-off particles and internal cracking clearly visible on the surface were also noted. The test was terminated after 25 freeze-thaw cycles. Durability of the exposed lime-pozzolan paste was determined from strength of the test prisms before and after the freeze-thaw cycling. The damage was evaluated in terms of decrease in compressive strength. The results of these tests are given in Table 7 and 8.

In general, the pastes cured at 70°C perform very well during the scaling tests. This is particularly true for those with lime/pozzolan ratios of 35 and 40% and water content of 45%, which show almost no deterioration after 25 cycles of freezing and

thawing. However, strength tests done after exposure indicate that there is a 15% reduction in compressive strength. On the other hand, the scaling resistance of pastes cured at 50 and 60 °C can be quite close to that of 70°C cured paste. This is clear from the test results for the pastes with 30 and 40% lime/pozzolan ratios and water contents of 45 and 50%. These pastes retain in excess of 67% of their strengths. As indicated in Table 7, some samples at 5 cycles show signs of internal cracking. This phenomenon only occurs in pastes with relatively low curing temperature. At 20 cycles, significant deterioration is observed in these samples. Some of the samples with a water content of 55% suddenly disintegrate at 25 freeze-thaw cycles. However, decreasing the water content from 55 to 45% positively influences all aspects of freeze-thaw behaviour of the pastes. This effect is much more pronounced on the pastes cured at 70°C. The ability of these pastes to retain their strengths is somewhat better than that of pastes cured at low temperature.

4 Conclusions

An experimental program was performed to examine the effect of curing temperature and matrix proportions on the strength of lime-natural pozzolan paste. The findings from this investigation are summarised as follows

Table 7: Durabilities of lime-pozzolan pastes after 25 freeze and thaw cycles

Curing temperature (°C)	Lime Pozz (%)	Water content (%)	Number of cycles				
			5	10	15	20	25
50	30	45	+	+	+	+	-
		50	+	+	+	-	-
		55	-	*	0	0	0
	35	45	+	+	+	+	-
		50	+	+	+	-	*
		55	+	-	*	0	0
	40	45	+	+	+	+	*
		50	+	+	+	-	*
		55	+	-	*	0	0
	60	45	+	+	+	+	-
		50	+	+	+	-	-
		55	+	+	+	-	*
60	30	45	+	+	+	+	-
		50	+	+	+	-	-
		55	+	+	+	-	*
	35	45	+	+	+	+	-
		50	+	+	-	*	0
		55	+	-	*	0	0
	40	45	+	+	+	+	-
		50	+	+	+	-	*
		55	+	-	*	0	0
	70	45	+	+	+	+	-
		50	+	+	+	+	-
		55	+	-	*	0	0
70	30	45	+	+	+	+	-
		50	+	+	+	+	-
		55	+	-	*	0	0
	35	45	+	+	+	+	+
		50	+	+	+	-	*
		55	+	+	-	*	0
	40	45	+	+	+	+	+
		50	+	+	+	-	-
		55	+	-	*	0	0

+ Sound
0 Scaling

- internal cracking
* internal cracking and scaling

0 Crumbling

Table 8: Relative strengths of lime-pozzolan pastes

Matrix ratios Lime Pozz. (%)	Water content (%)	Curing temperature (°C)		
		50	60	70
		σ_a / σ_b^*		
30	45	0.70	0.71	0.72
	50	0.68	0.74	0.75
	55	-	0.62	-
35	45	0.74	0.69	0.85
	50	0.64	-	0.62
	55	-	-	0.56
40	45	0.71	0.72	0.84
	50	0.67	0.69	0.74
	55	-	-	-

* σ_a / σ_b express the ratio of strengths after and before freeze and thaw cycles.

- 1) The effect of curing temperature differences (50-70°C) on strength yields similar results. However, curing at elevated temperature (120°C) has a great accelerating effect on the lime-pozzolan pastes.
- 2) The increase in water content from 45% to 55% causes a reduction in mechanical strengths, the magnitude of the decline being largely dependent upon the quantity of water in the mixture.
- 3) The setting time of pastes increases with an increase in lime and water content, ranging from 37 to 70 hours.
- 4) There is a strong statistical relationship between curing temperature and mechanical strength of pastes. Moreover, test results show a clear correlation between matrix proportions and mechanical behaviour. The proposed quadratic equations are found to be an accurate representation of these relations.
- 5) The pore structure of lime-pozzolan paste is basically formed by pores with size lower than 0.1µm.
- 6) The identified hydration products in the lime-pozzolan pastes are calcium silicate and gypsum.
- 7) The 70°C cured pastes with lime/pozzolan ratios of 35 % and 40 % and water content 45 % show excellent frost resistance. However, strength tests done after exposure indicate that there is a 15 % reduction in compressive strength.

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12 INFLUENCE OF EARLY AGE THERMAL CRACKING ON DURABILITY OF MASSIVE CONCRETE STRUCTURES IN MARINE ENVIRONMENT

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Abstract

The service life of massive concrete structures in marine environment can severely be reduced because of early age thermal cracking and subsequent acceleration of degradation. At the Magnel Laboratory for Concrete Research a durability evaluation project is going on concerning possibly cracked massive concrete structures in marine environment. Early age thermal cracking of massive concrete structures is studied by means of a newly developed simulation method, entirely based on the degree of hydration, including basic creep behaviour and basic shrinkage. The resulting cracks serve as input for a further durability evaluation. The influence of existing (thermal) cracks is studied experimentally by means of small-scale mortar specimens, provided with different sizes of cracks. An overview of the experimental program is presented. Small cracks do not seem to have a significant influence on carbonation. The chloride penetration however seems to be significantly influenced by the presence of cracks.

Keywords: concrete, massive structures, marine environment, durability, cracking

1 Introduction

The durability of concrete structures is a very important issue in actual concrete research. Especially concrete structures in marine environment seem to be prone to durability problems because of the presence of chlorides, leading to an important number of corrosion problems.

Concrete structures in marine environment often tend to be rather massive, like e.g. quay walls, locks and concrete armour units on breakwaters. The service life of these massive concrete structures can severely be reduced because of early age thermal cracking due to the heat of hydration, in combination with subsequent acceleration of degradation e.g. due to carbonation, chloride and sulphate diffusion and frost.

At the Magnel Laboratory for Concrete Research, University of Ghent, several research programs have been executed concerning massive concrete armour units for breakwaters, especially dealing with thermal stresses due to the heat of hydration (Dechaene et al 1994). A new simulation method for early age concrete thermal cracking has been developed, entirely based on the degree of hydration, including basic creep behaviour and basic shrinkage. A short overview of the simulation method is presented.

Furthermore an overview is given of the aims and methods of a durability evaluation project concerning possibly cracked massive concrete structures in marine environment. By means of some selected experiments the influence of existing thermal cracks on the further degradation of the concrete construction is examined. In the research project attention is also given to the interaction between different degradation mechanisms.

2 Early age thermal cracking

2.1 Heat of hydration and degree of hydration

The heat of hydration can be obtained experimentally in different ways. At the Magnel Laboratory isothermal hydration tests on cement paste and adiabatic hydration tests on concrete have been performed. In order to obtain adiabatic conditions a new test set-up was developed (De Schutter et al. 1992). Different types of cement were tested, resulting in a new general hydration model valid for Portland cement and blast furnace slag cement (De Schutter et al. 1995a). This model enables the calculation of the heat production rate as a function of the actual temperature and the degree of hydration, which is the main parameter used for the determination of the state of the hardening process. The hydration model can be implemented to calculate the evolution of temperature and of the stage of hardening in hardening concrete elements. It can also be applied for concrete containing admixtures like plasticizers and retarders (De Schutter et al. 1995b).

2.2 Thermal characteristics of hardening concrete

For the determination of the specific heat and the thermal diffusivity of hardening concrete, new tests methods were developed (De Schutter et al. 1995c). By means of these improved methods the evolution of the thermal characteristics during hardening was examined. It was concluded that the specific heat as well as the thermal diffusivity decreases linearly with increasing degree of hydration.

2.3 Calculation of the temperature field during hardening

The temperature field in hardening concrete elements can be calculated by solving the non-stationary Fourier equation with combined initial and boundary conditions. The boundary condition can be time-dependent applying a time-dependent convection coefficient or a time-dependent environmental temperature. The heat production and the thermal characteristics vary with the degree of reaction as explained in sections 2.1 and 2.2.

In order to verify the developed hydration model and thermal material laws, massive concrete cylinders were tested at the Magnel Laboratory for Concrete Research. A good agreement was found between the predicted and the measured temperatures (Dechaene et al. 1994). The calculated temperature fields can serve as a good basis for further stress calculations.

2.4 Mechanical behaviour of early age concrete

For the evaluation of the risk of thermal cracking in hardening massive concrete elements, knowledge of the development of strength, stiffness and deformability of early age concrete is extremely important. Based on an extensive experimental research program on hardening concrete elements, a degree of hydration-based description for the mechanical properties of the hardening concrete were elaborated (De Schutter et al. 1996a). E.g. for the compressive strength f_c the following equation was obtained:

$$\frac{f_c(r)}{f_c(r=1)} = \left(\frac{r - r_0}{1 - r_0} \right)^a \quad (1)$$

with $f_c(r)$ = compressive strength at degree of hydration r
 $f_c(r=1)$ = compressive strength at degree of hydration $r = 1$
 r_0, a = parameters depending on the concrete composition

Analogous degree of hydration based formulas were obtained for Young's modulus E_{c0} , tensile strength f_{ct} , Poisson's ratio ν and Strain at peak compressive stress ϵ_{c1} . An extension towards hardening concrete of the stress-strain relation for short-term loading given in the CEB FIP Model Code 1990 leads to a degree of hydration-based stress-strain relation. Good agreement with experimental results has been reported (De Schutter et al. 1996a).

Based on an extensive experimental program a new model was proposed for the basic creep of early age concrete. In this first model the degree of hydration is an important parameter influencing the creep evolution as well as the final creep value. In this way the fundamental nature of the degree of hydration in the early age basic creep behaviour was illustrated. The non-linearity of the basic creep at early age is taken into account (De Schutter et al. 1997a).

In a second model the degree of hydration is even more important, as basic creep is no longer described as a function of time but as a function of the evolution of the degree of hydration:

$$E_{cc}(r, r_b, \alpha_b) = E_{c0} \cdot \varphi_c(r, r_b) \quad (2)$$

$$\varphi_c(r, r_b) = c_1(r_b) \cdot \left(\frac{r - r_b}{1 - r_b} \right)^{c_2(r_b)}$$

with	ϵ_{cc}	=	basic creep strain
	ϵ_{c0}	=	instantaneous deformation at loading
	r	=	degree of hydration
	r_b	=	degree of hydration at loading
	α_b	=	stress level at loading
	c_1, c_2	=	parameters depending on r_b

In this way the final creep value as well as the creep evolution depends on the degree of hydration at loading r_b and on the stress level at loading α_b . The creep evolution depends on the evolution of the degree of hydration r .

For varying stresses an 'equivalent time method' can be applied, translated to a degree of hydration-based description. A good agreement with experimental results has been noticed (De Schutter 1996b).

2.5 Stress calculations

By means of the degree of hydration-based formulas for thermal and mechanical behaviour of hardening concrete, an accurate prediction of thermal stresses can be realised. This was verified by means of different test results obtained at the Magnel Laboratory for Concrete Research (De Schutter 1996b).

The stress calculation method, including basic creep behaviour, was also verified by means of test results obtained by T.N.O. (a Dutch research organisation) together with calculation results based on Van Breugel's relaxation formula for young concrete (Van Breugel 1980 and 1982).

During the tests carried out by T.N.O. hardening concrete prisms 150 mm x 150 mm x 800 mm were placed in a rigid frame in such a way that deformations of the test device caused by thermal dilatation of the hydrating concrete prisms could entirely be compensated. Dutch blast furnace slag cement HOC-A was used, which is very similar to the Belgian blast furnace slag cement CEM III/B 32.5 which was tested in the Magnel Laboratory for Concrete Research (De Schutter 1996b).

In figure 1 the experimentally obtained stress development is shown, together with some simulation results obtained by means of the degree of hydration-based calculation method, as well as the superposition method using Van Breugel's relaxation formula. It can be concluded that the degree of hydration-based calculation method enables an accurate simulation of the experimentally obtained stress evolution. As a comparison also some elastic calculations without creep are shown in figure 1.

2.6 Simulation of crack formation

For the experimental investigation of the evolution of the fracture energy during hardening, three-point bending tests were performed on unnotched prisms with an age ranging from 1 to 28 days. Three different cement types were considered. The development of fracture energy G_f during hardening can be described by a degree of hydration-based formulation analogous to equation (1) (De Schutter et al. 1997b).

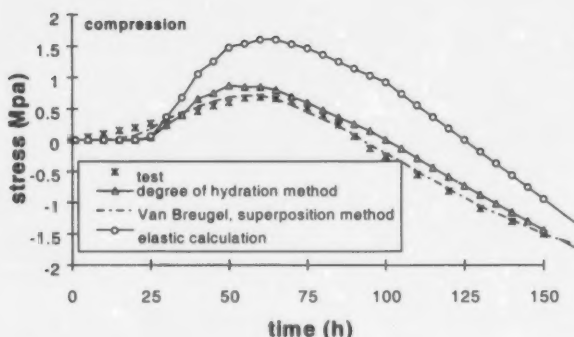


Fig. 1: Evaluation of the simulated stress results

Using the mentioned degree of hydration-based material laws a crack initiation and propagation simulation becomes possible. A research project is still going on at the Magnel Laboratory to implement the degree of hydration-based approach in a three-dimensional finite element program, including crack simulation. The obtained crack distribution and crack widths can serve as input for the further durability assessment of the massive concrete structures.

3 Durability assessment of cracked structures

3.1 Research project

A durability evaluation and simulation project concerning possibly cracked massive concrete structures in marine environments is going on at the Magnel Laboratory for Concrete Research. The main purpose of the project is a probabilistic formulation of service life of marine structures taking into account possible early age thermal cracking due to heat of hydration, and considering the interaction between different degradation mechanisms.

In a first stage of the project small scale experiments are carried out in order to study the influence of existing cracks on the carbonation and chloride penetration into concrete. The interaction between carbonation and chloride penetration is also investigated. In the sequel the experimental program is outlined, and some preliminary results are given.

3.2 Experimental program

In the experimental program carbonation and chloride penetration tests are executed on mortar specimens 40 mm x 40 mm x 160 mm, provided with different sizes of cracks, with a width up to 0.5 mm and a depth up to 10 mm. The specimens are made using traditional mortar moulds. The cracks are realised as notches by means of the positioning and removal afterwards of thin copper sheets inside the specimens.

Six different mortar compositions are investigated, as given in table 1. CEM I 42.5 R is a rapid hardening Portland cement, CEM III/B 32.5 HSR LA is a high sulphate resisting blast furnace slag cement with a limited alkali content, and CEM V/A 32.5 is a composite cement made of Portland clinker, blast furnace slag and fly ash. In some series, fly ash or silica fume is added. Normalised sand is used, as prescribed for traditional mortar tests.

Six different combinations of crack width and crack depth are considered, as given in table 2. Zero crack width and depth means that no crack is present.

Table 1: Mortar composition (mass parts)

Series	Cement	Puzzolan	Water	Sand
1	CEM I 42.5 R 1 part	none	• part	3 parts
2	CEM III/B 32.5 HSR LA 1 part	none	• part	3 parts
3	CEM V/A 32.5 1 part	none	• part	3 parts
4	CEM I 42.5 R 0.85 parts	fly ash 0.15 parts	• part	3 parts
5	CEM III/B 32.5 HSR LA 0.85 parts	fly ash 0.15 parts	• part	3 parts
6	CEM I 42.5 R 0.9 parts	silica fume 0.1 parts	• part	3 parts

Table 2: Combinations of crack width and crack depth

Crack depth	Crack width			
	0 mm	0.2 mm	0.3 mm	0.5 mm
0 mm	X			
5 mm		X	X	X
10 mm			X	X

During the first 28 days the mortar specimens are stored under water. At the age of 28 days an epoxy coating is applied at all surfaces, except on the surface containing the crack. In this way the protected surfaces are prevented from carbon and chloride penetration. Starting from 28 days the specimens are stored in different conditions as given in table 3. Five different treatments are considered: a 10% CO₂-environment, a 3.5% NaCl-solution, and combinations of CO₂-environment, NaCl-solution and water.

At regular times (8, 12, 16, ... weeks) the carbonation and the chloride penetration are examined experimentally. From each mortar specimen two slices of thickness 1 cm are sawn. The first slice is sprayed with a phenolphthalein-solution in

order to determine the carbonated zone. The second slice is sprayed with a 0.1 N aqueous solution of silver nitrate in order to determine the depth of penetration of chloride ions. According to Otsuki et al. (1992) the (soluble) chloride ion concentration at the colour change border is recognised to be of the order of 0.15% by weight of cement.

After sawing the two slices the remaining mortar specimen is covered again with the carbon and chloride resisting epoxy coating (except for the 'cracked' surface), and the treatment according to table 3 is continued.

3.3 Preliminary results

At the age of 28 days the density, strength and porosity (water absorption) of the different mortar compositions are determined. The results are given in table 4. For each specimen the carbonation depth and the chloride penetration are determined as described above. At this moment results have been obtained for series 1 to 4, for an age ranging from 8 weeks to 12 weeks. Figure 2 e.g. shows some carbonation profiles for series 1, whereas figure 3 shows the chloride penetration for series 2.

3.4 Preliminary conclusions

Based on the preliminary results for series 1 to 4 some preliminary conclusions can be drawn:

- The presence of cracks with a crack width of maximum 0.3 mm seems to have no significant influence on the carbonation depth. A crack width of 0.5 mm seems to increase somewhat the local carbonation depth.
- The chloride penetration is significantly influenced by the presence of cracks, even for the smaller crack widths.
- The chloride penetration seems to decrease somewhat the carbonation depth.
- The carbonation seems to increase somewhat the chloride penetration.

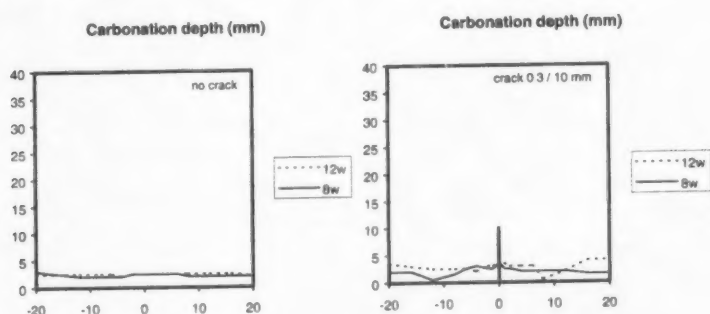
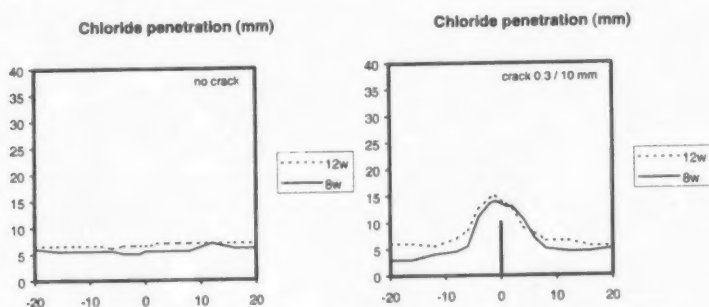
A more detailed evaluation and justification of these preliminary conclusions will be done using the forthcoming test results.

Table 3: Treatments

Code	Treatment
A	Permanent 10% CO ₂ -environment
B	Permanent 3.5% NaCl-solution
C	Cycle 1 week 10% CO ₂ -environment / 1 week 3.5% NaCl-solution
D	Cycle 1 week 10% CO ₂ -environment / 1 week water
E	Cycle 1 week water / 1 week 3.5% NaCl-solution

Table 4: Density, strength and porosity at 28 days

Series	Density (kg/m ³)	Bending strength (N/mm ²)	Compressive strength (N/mm ²)	Porosity (%)
1	2255	8.65	57.2	8.8
2	2280	10.75	46.2	9.3
3	2240	6.75	46.4	9.1
4	2255	8.40	50.3	9.2
5	2260	8.90	43.2	9.4
	2240	9.90	65.2	9.1

**Fig. 2: Carbonation depth for some specimens of series 1 treatment A****Fig. 3: Chloride penetration for some specimens of series 2 treatment B**

4 Conclusions

At the Magel Laboratory for Concrete Research a new simulation method for early age thermal cracking has been developed, entirely based on the degree of hydration, including basic creep behaviour and basic shrinkage. An overview of the simulation method is presented. Furthermore a short overview is given of the aims and methods of a recently started durability evaluation project concerning possibly cracked massive concrete structures in marine environments. The experimental program is outlined, and some preliminary conclusions are drawn. The presence of cracks seems to have more influence on the chloride penetration than on the carbonation.

5 Acknowledgement

The author is Postdoctoral Research Fellow of the Fund for Scientific Research, Flanders (Belgium) (F.W.O.-VL). The support of the Fund for Scientific Research is greatly appreciated.

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13 AVOIDING THE THAUMASITE FORM OF SULFATE ATTACK

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Abstract

Recent site and laboratory investigations carried out by BRE on problems of sulfate attack have shown that the mineral thaumasite has been responsible for the deterioration of concretes and mortars specifically designed to give good sulfate resistance. It has been shown that thaumasite may form if concretes containing commonly used limestone aggregates are exposed to sulfate solutions. Therefore, among other measures there is a need to establish a type of carbonate aggregate concrete which can resist this form of sulfate attack. Previous work carried out by BRE indicated that the use of blastfurnace slag cement could improve the resistance of carbonate aggregate concretes. In order to examine this possibility further sulfate resistance tests have been carried out at 5°C and 20°C on concretes containing various limestone aggregates. The main cementitious materials used with these aggregates were ordinary Portland cement (OPC), sulfate resisting Portland cement (SRPC), and an ordinary Portland cement blended with ground granulated blastfurnace slag (OPC/ggbs). This paper describes the one and two year results for these sulfate resistance tests. The results found that the ggbs performed satisfactorily with the good quality limestone aggregates; the concrete containing ggbs/Carboniferous limestone performed extremely well under the most aggressive conditions, and under the same aggressive conditions, the ggbs concretes did not perform satisfactorily when combined with the lower grade limestone aggregates. The SRPC concretes, stored in medium to strong magnesium sulfate solutions at 5°C, performed satisfactorily at 1 year with all grades of aggregate. However after two years, they have started to show unsatisfactory sulfate resistance with the magnesian limestone and lower grade calcareous limestone aggregate. This delay in deterioration of SRPC concretes has been seen in other sulfate resistance test work at BRE. The ggbs concretes containing good quality limestone aggregate continued to perform better than the equivalent OPC concretes.

Key words: Thaumasite, carbonate aggregate, sulfate attack, SRPC, GGBS.

Durability of Building Materials and Components 8 (Volume One). Edited by M.A. Lacasse and D.J. Vanier. Published in 1999 by NRC Research Press, Ottawa ON K1A 0R6, Canada. ISBN 0-660-17737-4

1 Introduction

Recent evidence has been found to suggest that concretes containing commonly used limestone aggregate sometimes fail to perform satisfactorily when exposed to sulfate bearing groundwater (Crammond and Nixon, 1993, Crammond and Halliwell, 1995). This can be the case even when a sulfate-resisting Portland cement (SRPC) is used. The extent of this form of sulfate attack in the field is unknown. Preliminary site and laboratory investigations carried out by BRE (Crammond and Nixon, 1993, Crammond and Halliwell, 1995) on problems of sulfate attack have shown that the formation of the mineral thaumasite ($\text{CaSiO}_3 \cdot \text{CaCO}_3 \cdot \text{CaSO}_4 \cdot 15\text{H}_2\text{O}$) has been responsible for the deterioration of these concretes.

From these studies it has been shown that the conditions favoured for thaumasite formation are cold wet environments, a source of calcium silicate, and a readily available source of carbonate and sulfate ions. The formation of thaumasite has a different effect on concretes and mortars compared with the classical form of attack in which ettringite is produced. The formation of thaumasite involves attack on the calcium silicate hydrate phase (CSH), which is the main binding agent in cements. Thaumasite formation is therefore accompanied by a reduction in the binding ability of the CSH phase resulting in a loss of strength and transformation of the concrete or mortar into a mushy incohesive mass.

Previous laboratory work carried out by BRE (Crammond and Halliwell, 1995) suggested that improved resistance against the thaumasite form of sulfate attack may be achieved by using a blended cement containing blastfurnace slag (ggbs) as the cementitious constituent. The work described in this paper explores the prospect of using ggbs cement as an effective sulfate-resisting cement in concretes containing limestone aggregate that are exposed to sulfate-bearing groundwater.

2 Concrete mixes

This work examined the performance of a series of concrete mixes designed for good sulfate resistance as specified in Digest 363 (BRE 1996). The mixes contain various carbonate-containing aggregates together with a range of cement types as follows:

- **Cement Types.** The cements used were: PC (7.2% C_3A), SRPC (0% C_3A), 30% PC with 70% ggbs of normal (12%) alumina content, 30% PC with 70% ggbs of high (17%) alumina content and BRECEM (50% calcium aluminate cement (CAC) and 50% ggbs). Halliwell and Crammond, 1997 gives a complete list of the chemical analyses of the cementitious materials.
- **Aggregates.** The aggregates used in this study were split into two main groups. Firstly, good quality aggregates normally used in concretes. These were; (i) Thames Valley Flint, (ii) a mixture of crushed Carboniferous limestone coarse and Jurassic limestone fine aggregate to produce an all-carbonate aggregate concrete and (iii) a mixture of Thames Valley flint coarse

and crushed Carboniferous limestone fine aggregate is needed produce an aggregate with carbonate only in the fine fraction. Secondly, two further aggregate combinations were included. These were materials which would not normally be used in concrete because of their high absorptivity and/or low strength and these were: (i) a mixture of poor quality magnesian carbonate as coarse and fine aggregate and (ii) a mixture of inferior oolitic limestone as coarse and fine aggregate. A complete list of the physical properties of the aggregates are given in Halliwell and Crammond, 1997.

2.1 Concrete mix designs

All mixes were designed to have a free water/cement ratio of 0.50, which is the maximum for Class 3 sulfate conditions in accordance with BRE Digest 363 (BRE, 1996). For the 'core' mixes made with the normal concreting aggregates, the cementitious content was fixed at 350 kg/m³. In order to achieve a compactable concrete with a w/c of 0.50 for the mixes made with the low-quality aggregates, it was necessary to increase the cementitious content to 400 kg/m³ and use a water-reducing admixture. Even with these adjustments, the inferior oolitic mixes were of low workability.

Details of the compositions of the mixes, their compacting factors and 28-day strengths are provided in reference 6. For the six 'core' mixes, triplicate specimens were exposed in each test condition, requiring 75 cubes per mix. For the eleven smaller mixes, duplicate specimens were used for each test condition with some of the test permutations being omitted, requiring only 36 cubes per mix. The total programme involved nearly a thousand cubes.

3 Experimental procedures

3.1 Preparation and storage of specimens

All concretes were vibrated into 100mm cube moulds and stored for 24 hours under damp hessian and polythene sheet, before being de-moulded and numbered. The cubes were then cured for a further 27 days at 20°C, either in water or in air at 65% relative humidity. For both air and water storage, the cubes were stacked three high. Some of the cubes stored in water stuck together and had to be prised apart. Immediately after curing, the cubes were transferred into containers containing the test solutions. Five concentrations of solution were used:

- Solution 'E': magnesium sulfate, of strength 1.8% as SO₄
- Solution 'C': magnesium sulfate, of strength 0.42% as SO₄
- Solution 'M': magnesium sulfate, of strength 0.14% as SO₄
- Solution 'I': sodium sulfate, of strength 1.8% as SO₄
- Tap water

Each container held four cubes, spaced apart in a single layer, and submerged in approximately 3.5 litres of solution. The 'control' cubes, which contained no carbonate aggregate, were kept separate from cubes containing carbonate aggregate.

To simulate the static conditions characteristic of a clay soil on site, some of the solutions were never changed. In other containers the solutions were emptied and replaced with fresh solutions every 3 months, to simulate a mobile groundwater. A further variable in the test conditions was temperature; specimens were tested in solutions maintained at 5°C and at 20°C.

3.2 Visual and photographic assessment

After approximately one and two years, the cubes were visually assessed and photographs were taken of specimens that showed evidence of attack.

3.3 Wear rating

'Wear rating' is a measure of the attack on the corners of a cube and was measured after approximately one and two years. Previous experience at BRE (Harrison, 1985) has found it to be a useful measurement on specimens subject to conventional sulfate attack, because here the initial damage is predominantly cracking or erosion of the corners. Measurements are made on the struck face and the opposite face of the cubes. On each of these faces two diagonal measurements are made of the distance from the edge of one corner damage to the edge of the diagonally opposite corner damage. Knowing the corresponding length on an undamaged cube, the wear rating of a cube is calculated as: "the sum of the loss in millimetres of the four measured diagonals, divided by 8" (Harrison 1985), i.e. the average depth of erosion or damage for one corner (in millimetres).

3.4 XRD studies

Any interesting surface deposit or deterioration product was removed and analysed using X-ray diffraction, (XRD). The samples were allowed to dry naturally and were then finely ground for qualitative XRD using a Siemens D500 automated diffractometer.

4 Presentation of results

A combination of wear rating values (as shown in Table 1) and visual assessment (photographs shown in Figure 1) gave a very good indication of the degree of sulfate attack in the cubes under test. Little deterioration was detected at 20°C and consequently this paper only reviews the 5°C data. All wear rating tables and photographs are presented in Halliwell and Crammond 1997.

4.1 Assessment of sulfate-resisting properties using the wear rating results

It has been shown that normal production, demoulding and handling of cubes produces 'wear rating' values due to imperfect cubes of between 1 and 3 with a mean of 2 (Harrison, 1985). Harrison differentiated between the wear ratings obtained from the struck face and the opposite face. He detected greater attack at the corners of the struck face and deduced that wear rating values of greater than 5 (2+3) in the first year could be taken as an indication of the onset of sulfate attack. Hence values between 2 and 5 for the struck face would indicate satisfactory resistance to sulfate attack.

Table 1: Wear rating for 100mm cubes after two years storage in sulphate solutions at 5°C

Concrete Type	Concrete Class	Aggregate type	Sulphate solution type								
			1.8% SO ₄ Mg/SO ₄ (change 3M) no change		1.8% SO ₄ Na ₂ SO ₄ (change 3M) no change		0.42% SO ₄ Mg/SO ₄ (change 3M) no change		0.14% SO ₄ Mg/SO ₄ (change 3M) no change		
OPC	350	Thames flint	11.0	10.0	13.0	12.0	8.0	8.0	1.0	0.5	0.0
OPC	350	Carbonif Lst	18.0	18.0	8.0	5.0	8.0	4.0	1.0	1.0	0.0
OPC	400	Carbonif Lst	19.0	13.0	1.5	10.0	1.5	10.0	1.0	2.0	0.0
OPC	400	Magnesian limestone	13.0	8.0	11.0	3.0	8.0	2.0	4.0	0.0	0.0
OPC	350	Flint	17.0	9.0	8.0	3.5	8.0	2.0	4.0	0.0	0.0
SLAG/OPC	350	Thames flint	14.0	3.5	10.0	0.0	5.5	0.0	0.0	0.0	0.0
SLAG/OPC	350	Carbonif Lst	3.5	3.5	0.5	0.0	0.0	0.0	0.0	0.0	0.0
SLAG/OPC	400	Inferior Oolitic Limestone	29.0	10.0	24.0	9.0	15.0	0.0	0.0	0.0	0.0
SLAG/OPC	400	Magnesian limestone	15.0	6.0	13.0	4.0	11.0	1.0	0.0	0.0	0.0
SLAG/OPC	350	Flint	2.5	0.0	1.0	0.0	0.0	0.0	0.0	0.0	0.0
SRPC	350	Thames flint	4.0	2.0	7.0	1.5	2.5	0.0	0.0	0.0	0.0
SRPC	350	Carbonif Lst	9.0	6.5	5.5	0.0	8.0	0.0	4.0	0.0	0.0
SRPC	400	Inferior Oolitic Limestone	12.0	2.5	4.0	0.0	9.0	0.0	0.0	0.0	0.0
SRPC	400	Magnesian limestone	12.0	3.0	4.5	1.0	10.0	0.0	4.5	0.0	0.0
SRPC	350	Flint	2.5	3.5	3.0	1.5	1.5	3.0	0.5	0.0	0.0
SLAG/20OPC	350	Thames flint	25.0	13.0	5.0	3.0	13.0	0.0	1.5	0.0	0.0
SLAG/20OPC	350	Carbonif Lst	7.0	1.5	1.0	0.0	0.0	0.0	1.5	0.0	0.0
BRECEM	350	Thames flint	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
BRECEM	350	Carbonif Lst	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
As cured											
OPC	350	Thames flint	1.5	2.0	1.0	0.0	-	-	1.0	0.0	0.0
OPC	350	Carbonif Lst	14.0	11.5	6.0	3.0	-	-	2.0	0.0	0.0
SLAG/OPC	350	Thames flint	0.0	0.0	0.0	0.0	-	-	0.0	0.0	0.0
SLAG/OPC	350	Carbonif Lst	0.0	0.0	0.0	0.0	-	-	0.0	0.0	0.0
SRPC	350	Thames flint	5.5	2.0	4.5	3.0	-	-	0.0	0.0	0.0
SRPC	350	Carbonif Lst	0.0	3.0	3.5	0.0	-	-	0.0	0.0	0.0

values > 8 imply poor sulphate resistance (bold) values < 8 implying satisfactory sulphate resistance

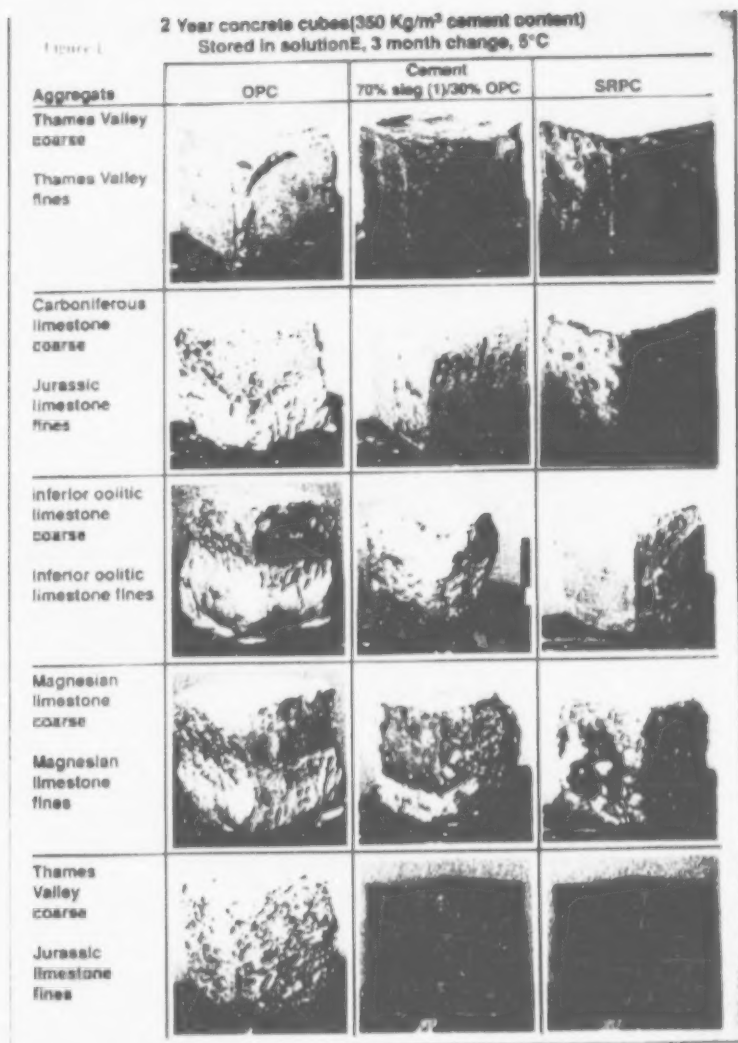


Fig. 1: Sulfate attack in the cubes under test

In the present study, the general impression was that the wear rating values measured for the struck face and the opposite face were similar. The opposite face was therefore treated as being just as susceptible to attack as the struck face and Harrison's performance criteria have been adopted as follows:

one year criteria

values > 5 imply poor sulfate resistance

values $\leq$ 5 imply satisfactory sulfate resistance

values $\leq$ 2 imply good sulfate resistance

two year criteria

values > 8 imply poor sulfate resistance

values $\leq$ 8 imply satisfactory sulfate resistance

values $\leq$ 4 imply good sulfate resistance

5 Discussion of the results from the sulfate resistance tests

The main finding from the 2 year cube assessment data was that concretes made with BRECEM performed the best, followed by those containing ggbs and SRPC cements with the OPC concretes showing the worst performance. It is important to note that at 1 year the SRPC concretes made with poor quality limestone aggregates were performing well, but after 2 years they had developed an unsatisfactory wear rating.

Two other important findings were that the sulfate resistance of the concrete specimens continued to be much reduced at 5°C and, at this lower temperature, it was predominantly concretes containing limestone-bearing aggregate which deteriorated.

Concrete cubes containing good quality carbonate aggregates performed well at 5°C when used in combination with all cement types except OPC. No concretes performed satisfactorily when they contained the poorer quality aggregates. At 1 year it was found that the SRPC was performing well with the poor quality aggregates, but at 2 years this was found not to be the case.

The ggbs concrete has performed particularly well when made with Carboniferous coarse and Jurassic fines. This combination performed better than the equivalent SRPC concretes, as shown in Figure 1 and Table 1. In this study, the most aggressive solution was found to be the strong magnesium sulfate solution, followed by the strong sodium sulfate solution. The medium magnesium sulfate solution gave some deterioration but only really with the concretes that had performed very poorly in the stronger solutions. One exception is the poor performance of the SRPC concretes after 2 years exposure to solution C at 5°C compared with their satisfactory performance after one year. No significant sulfate attack was found in the very weak magnesium sulfate solution. On the whole the solutions that were changed every three months caused more deterioration and attack than those that were left unchanged.

The effect of air-curing the concretes was to enhance significantly the sulfate resistance of all the concretes. Even the poorest quality concrete subjected to the most aggressive environment benefited from this form of curing. However, in the

case of OPC concrete containing Carboniferous limestone coarse aggregate the performance of the air-cured specimens deteriorated greatly between one and two years (see Figure 2). This was due to preferential attack on their 'stuck faces' (see later section).

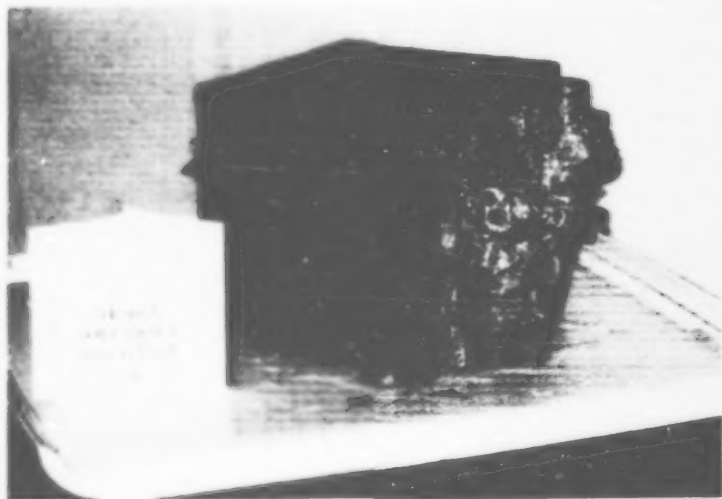


Fig. 2: Air cured OPC/Carboniferous limestone cube stored in solution E at 5°C. The effect of the stuck faces during curing is clearly illustrated

After the one-year assessment some of the results disagreed with data obtained from a previous study (Crammond and Halliwell, 1995, 1996). The two main differences were that:-

1. Concrete mixes containing SRPC and Magnesian limestone deteriorated in the earlier work but performed well in the present investigation.
2. Concrete mixes containing SRPC and either Carboniferous limestone or Jurassic limestone were showing definite signs of wear after one-year of exposure in the first programme of work but performed satisfactorily after 14 months exposure in the current study.

However, after two years it has become apparent that the present work does agree with the earlier studies. Firstly, concrete mixes containing SRPC and Magnesian limestone have now deteriorated to unsatisfactory levels. Secondly the SRPC concretes made with Carboniferous limestone are now showing signs of deterioration.

The reason for these differences could be the slight variations in composition between the two SRPCs and the two Magnesian limestones used in the two studies. It is known that different SRPCs, for example, may remain dormant for different lengths of time before the onset of attack. The SRPC used in

this work contains a slightly lower C_3A content compared with that used in the previous work (Crammond and Halliwell 1995) which may have provided an improved sulfate resistance.

Inferior oolitic limestone was not included as one of the aggregates examined in the first programme of work. The poor performance of OPC concretes containing the inferior oolitic limestone in the present study was expected but the poor performance of the equivalent cubes containing ggbs was not.

Thaumasite was found by X-ray diffraction studies to be the main reaction product in the concretes that had deteriorated at the lower storage temperature of 5°C.

During the curing of many of the samples, some of the cubes stacked on top of each other became bonded together. Many of these faces which were 'stuck' to each other had to be prised apart before immersion in the sulfate solutions. Some of the concretes were air-cured after demoulding for a period of 28 days and subsequently placed in sulfate solutions. The effect of the cubes being 'stuck' during curing was dramatically shown in these air-cured mixes. Figure 2 shows the effect of an OPC concrete with Carboniferous aggregate stored in the most aggressive environment (solution E, strong $MgSO_4$, at 5°C). No significant corner wear was recorded on this cube after one year exposure, but where the faces had 'stuck' together, they had literally burst open. After 2 years storage in solution E, the wear rating values of this particular cube had increased to a unsatisfactory level. The reason for this mode of attack appeared to be that, as a result of these cube faces being stuck together during curing, calcium hydroxide from the hydrating cement would not have been carbonated during air-curing or leached during water-curing. Consequently this retained calcium hydroxide has been readily available for reaction with sulfate ions.

6 Conclusions

- Ggbs concretes performed well when made with the good quality carbonate aggregates. Apart from the equivalent BRECEM concrete the mix containing ggbs and Carboniferous limestone was the best performing concrete under the most aggressive environment (solution E at 5°C). Ggbs concretes performed less satisfactorily when they contained the lower grade inferior oolitic and Magnesian limestone aggregates.
- The SRPC concretes made with the poor quality limestones performed satisfactorily up to one year but by two years were deteriorating significantly. SRPC concretes performed satisfactorily with both the good quality limestones and flint aggregates at one year, but were showing early signs of deterioration at 2 years with the good quality limestones.
- Deterioration was much more noticeable in the concrete cubes stored at 5°C than those stored at 20°C and at the lower temperature, thaumasite was identified by XRD studies to be the main reaction product formed.
- As expected, OPC concretes were the least resistant against sulfates. The worst sulfate resistance was found with OPC concretes exposed to strong magnesium sulfate solutions. Those containing carbonate-bearing aggregates had severely

eroded whereas those containing flint aggregate had remained intact but had expanded significantly.

- All the BRECEM concretes performed perfectly in every environment.
- Most air-cured concretes performed extremely well, even concretes made with OPC and Inferior Oolitic limestone stored at 5°C. Problems only arose when excess erosion occurred at 'stuck faces' which had not therefore been exposed to the air.
- Storage in the strong magnesium sulfate solution produced the greatest degree of sulfate attack followed by storage in the strong sodium sulfate solution.
- In general immersion in solutions that were changed every three months resulted in more deterioration and attack compared with those that were left unchanged.

7 Acknowledgements

The work described has been carried out under the Partners in Technology scheme, and was jointly funded by the Department of the Environment, Transport and the regions (DETR) and the Cementitious Slag Manufacturers Association (CSMA).

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14 FIRE DURABILITY OF VOLCANIC PUMICE CONCRETE WITH SPECIAL REFERENCE TO THIN WALLED FILLED SECTIONS

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Abstract

This paper concentrates on the durability of volcanic pumice concrete (VPC) subjected to fire or elevated temperature. Performance is judged based on strength and physical changes of VPC specimens as well as thin walled composite (TWC) filled sections using VPC as in-fill. Investigation suggests that it is possible to produce a VPC that is 30% lighter than normal concrete and of satisfactory strength. Both VPC and TWC specimens are subjected to fire for different durations, maximum one hour. It is found that the strength of both types of specimens decreases with the increase of the duration of fire. The fire resistance of VPC in TWC section is found to be better than its individual fire resistance. Investigation is in progress towards the development of a design specification for fire resistance of VPC and TWC sections.

Keywords: Volcanic pumice, thin walled composite, fire, strength, light weight

1 Introduction

Pumice is a natural material of volcanic origin produced by the release of gases during the solidification of lava. The cellular structures of pumice is created by the formation of bubbles or air voids when gases contained in the molten lava flowing from volcanoes become trapped on cooling. The cells are elongated and parallel to one another and are sometimes interconnected. Volcanic pumice (VP) has been used as aggregate in the production of light weight concrete in many countries of the world. So far, the use of pumice was dependent on availability and limited to countries where it is locally available or easily imported. Neville (1981) reported that satisfactory concrete, which is 2 to 3 times lighter than

normal concrete, having good insulating characteristics with high absorption and shrinkage, can be manufactured using volcanic pumice.

Volcanic activities are a common phenomena for a country like Papua New Guinea, and due to frequent volcanic eruption, volcanic debris such as volcanic ash (VA) and pumice are found abundantly (Hossain 1998a). The 1994 volcanic eruption that occurred in the East New Britain province was the second destructive one in history, which devastated the province and created an environmental disaster. A comprehensive programme of research is put forward with motivation from local cement and construction industries in an attempt to explore the possible utilization of VA and VP in concrete production, which can not only provide low cost cement and concrete, but also helps to decrease environmental hazard. The research is conducted based on Papua New Guinea and local structural needs, so that its benefits can be utilized in low cost construction suitable for the ordinary people.

Current research by Hossain (1998b) included investigations on the possible use of VPP (volcanic pumice powder) as cement replacement material. The results suggested that the normal consistency and setting time of Portland cement are affected by the replacement of cement by VPP. Manufacture of blended PVPC (Portland volcanic pumice cement) similar to PFAC (Portland fly ash cement) of type FC (according to Australian Standard: AS1317, 1982) is possible with maximum replacement up to 20%. A possible use of this PVPC will be in mass concrete construction, due to lower heat of hydration and higher setting time compared to Portland cement. Results also showed that by using 35% VPP, it is possible to obtain a mortar having a strength of 24 MPa (28 day) using locally available sand which is acceptable for normal construction use in the context of Papua New Guinea.

This paper will present the properties of volcanic pumice concrete (VPC) using VP as coarse aggregate (VPA), and especially its performance in thin walled composite (TWC) filled sections. The use of TWC sections is a new idea for beams and columns (Hossain 1998c; Hossain and Mol 1998; Hossain, Mol and Wright 1998), comprising cold formed steel elements with an infill of concrete that is suitable as replacement for hot-rolled steel or reinforced concrete in small to medium sized buildings. Typical TWC beams and columns are shown in Fig. 1.

The inherent advantages of this system are derived from its structural configurations. TWC sections do not require temporary form work for infill concrete, as the steel acts as form work in the construction stage and as reinforcement in the service stage. Advantages of this system also include a very light construction weight, excellent surface finish, relatively slender dimensions, enhanced ductility and the potential for semi-rigid connections. The infill concrete in TWC sections is less likely to be affected by adverse temperature and winds as experienced in the case of reinforced concrete. The in-fill concrete is generally cured quickly, and in any case, the load capacity of the steel alone may be relied upon for most construction loads.

TWC sections are more susceptible to fire, although the thermal mass of the concrete in-fill provides reasonable protection to most fire loads. This paper will mainly focus on the fire resistance of VPC in general and in TWC sections in particular.

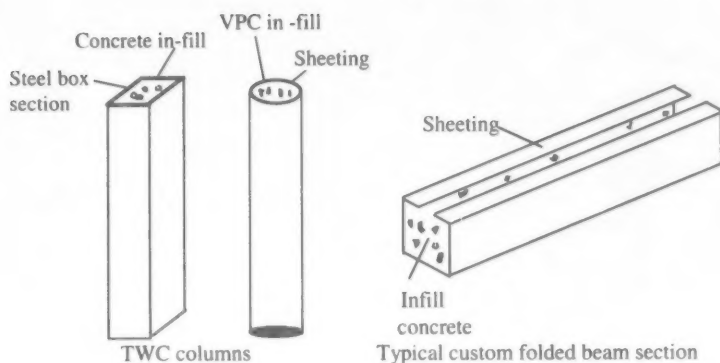


Fig. 1: Schematic of TWC sections

2 Research significance

Research is now ongoing to investigate the potential of manufacturing VPC, and its use as light weight concrete in-fill in TWC sections. The improved performance of VPC in a confined environment will allow the use of comparatively low strength VPC in this form of construction, as the main function of the concrete in-fill is to prevent local buckling of the thin sheeting. With the use of VPC, weight can be reduced by a further 35% compared to normal concrete.

The building of an elegant structure, satisfying structural (strength associated with ductility and lightness), construction (feasible and simple) and economic (low cost) requirements, is vital for Papua New Guinea, and TWC section can meet these requirements. The viability of this form of construction should be assessed in light of structural performance, feasibility in fabrication, and construction and economy. For feasibility assessment, the co-ordination between manufacturing and construction industry is needed. With this system, the steel strip is reasonably cheap to import and the cost of setting up a rolling mill is acceptable in quite poor countries. The manufacturing of sheeting locally and use of locally available VP, will provide economy. The completion of this project will provide a foundation for the use of VPC and TWC sections in the construction industry of Papua New Guinea.

3 Experimental study

A series of tests were carried out on the use of VP as a cement replacement material and as a coarse aggregate in cement and concrete. The investigation was based on VP obtained from Tavurvur and Vulcan craters located in the Rabaul area of the East New Britain province of Papua New Guinea. A brief summary of the findings will be included in this paper as a background investigation.

3.1 Background investigation

Tests were conducted (Hossain and Kaio 1998) to investigate the effect of the replacement of normal coarse aggregate by different percentages of volcanic pumice aggregate (VPA) on the strength, modulus of elasticity and density of resulting volcanic pumice concrete (VPC). Typical concrete mix designs for VPC (using 100% VPA as coarse aggregate) and representative normal concrete (NC) using 100% normal stone aggregate are presented in Table 1. The coarse aggregate consisted of 10mm maximum crushed gravel (for NC) and VPA (for VPC), with river sand as fine aggregate. The particle size distribution of aggregates performed according to AS (Australian Standard) 1289.C6.1-1977 are presented in Fig. 2. VPA was found to be 3.24 times lighter than normal stone aggregate. Higher degree of porosity in VPA lead to almost 13 times higher water absorption than normal stone aggregate. Locally manufactured Portland cement called 'Paradise' was used in the concrete mixes.

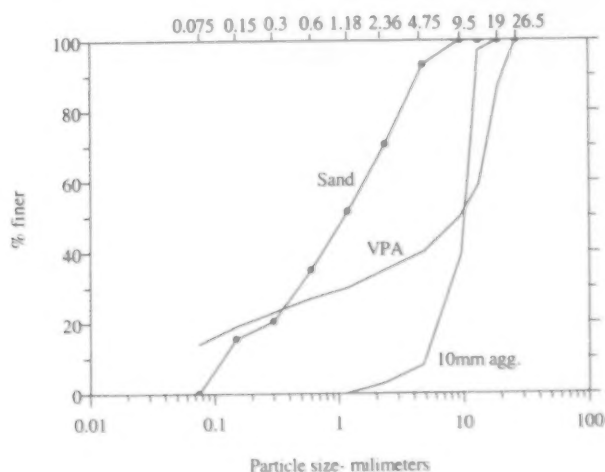


Fig. 2: Particle size distribution

Table 1: Concrete mix designs

Mix designation	VPA kg/m ³	Normal aggregate kg/m ³	Sand kg/m ³	Cement kg/m ³	NW/C	TW/C
VPC	412	0	704	426	0.46	0.82
NC	0	1372	704	426	0.36	0.48

NW: Net water; TW: Total water including absorbed water; C: Cement

The strength, modulus of elasticity and density of VPC and NC are compared in Fig.3. Results indicated that by using 100%VPA (designated as VPC), it is possible to obtain a VPC of 18 MPa (28-day cylinder strength: mix 1:2:4) and can produce a VPC that is 25% lighter than the normal concrete (NC: 0% VPA). The strength and modulus of elasticity (E) of VPC decreased with the increase of the percentages of VPA in the mix.

3.2 Investigations

The investigation concentrated on the performance of VPC under fire loading conditions. The performance was judged based on strength and physical changes of the VPC (only concrete) and TWC test specimens. The VPC mix used in the manufacture of test specimens in this investigation was similar to that described in Table 1. Two series of tests were conducted with detailed shown in Table 2.

Series A concentrated on normal VPC specimens with standard 150 mm x 300mm cylinders as pure concrete test specimen. A total of 21 specimens were cast and cured under water before subjected to fire.

Series B included a total of 63 specimens consisting of 42 TWC columns with VPC in-fill and 21 VPC in-fill (only concrete having same dimensions as in-fill concrete in TWC columns) column. The specimens (Fig. 4) were 75 mm diameter circular columns having a height of 300 mm. TWC columns were manufactured by using thin steel sheeting of 0.4 mm thickness.

Table 2: Test program

Type	Specimen Designation	Features	No. of tests
Series A (150mm x 300mm)	Cylinder	only VPC	21
Series B Cylindrical (75mm x 300 mm)	TWC column	Steel & VPC in-fill	21
	TWC (VPC in-fill column)	Steel & VPC in-fill	21*
	VPC column	only VPC	21

* subjected to fire as TWC but steel sheet is removed before tested under axial load

3.2.1 Testing and observation

Specimens were cured in water for 60 days and then air dried before being subjected to fire in a furnace maintained at a fixed temperature of 600°C. The specimens were subjected to fire for different time duration's, ranging from 0 to 60 minutes, with an increment of 10 minutes. For each time duration, three specimens were tested to calculate mean test results with reasonable accuracy.

The test specimens were removed from the furnace after the specified duration of heating and then allowed to cool to room temperature before being tested in the compression testing machine. For VPC infill specimens, the steel sheeting was removed before they were tested. The rate of loading was kept constant for each test specimen. The specimens were loaded to failure and associated ultimate load with modes of failure was observed.

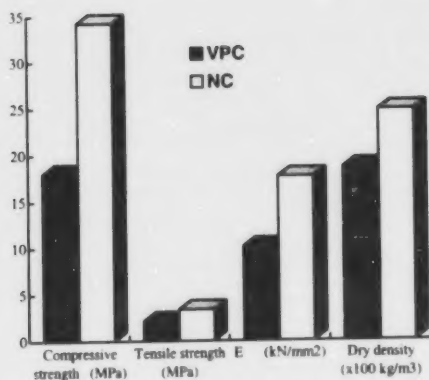


Fig. 3: Comparative study of VPC and NC

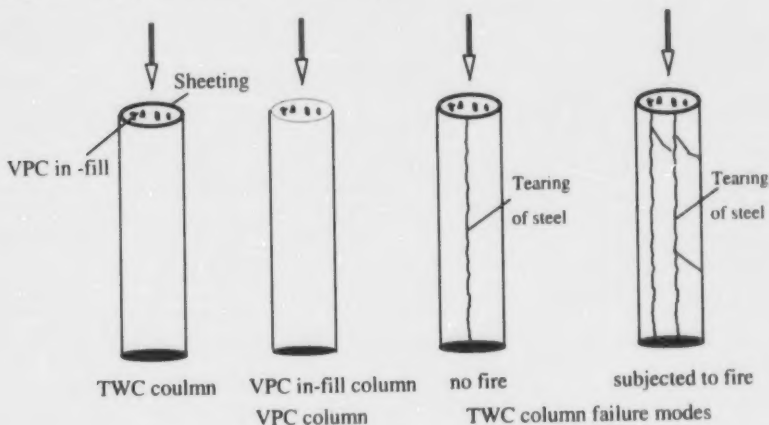


Fig. 4: Test specimens (Series B) and failure modes

The failure of the TWC columns (not subjected to fire) were found to be due to tearing of the thin steel plate along a diametrical plane (Fig. 4). This was due to the resulting hoop stress in steel and no local buckling was formed. This confirms the fact that the steel plate should be of sufficient thickness to develop local buckling with subsequent attainment of plastic yielding in the steel plate to further enhance the strength of the columns. For the columns subjected to fire, the tearing pattern of the steel sheet changed with increasing duration in the fire. Instead of a single diametrical tearing, the sheet teared in several diametrical planes as shown in Fig. 4. This may be due to the reduction of strength of the sheeting due to heating.

The colour of the volcanic pumice aggregate was changed by the fire exposure to black, as observed from the tested VPC and TWC specimens, and no spalling of concrete was observed in VPC specimens.

4 Results and discussion

The performance of VPC under fire will be described based on test results.

4.1 Performance of cylinder VPC (Series A) specimens

The compressive strength of cylinder specimens decreases with increasing duration of fire (Fig. 5). The strength is decreased by about 53% as the duration of fire is increased from zero to one hour.

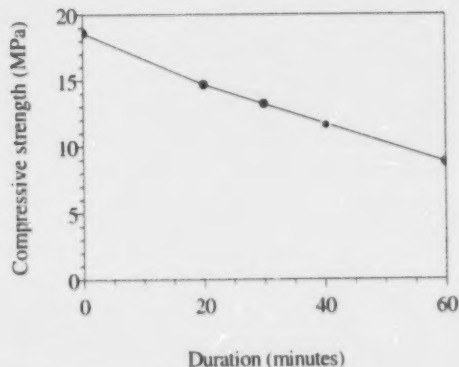


Fig. 5: Effect of fire on compressive strength of VPC

4.2 Performance of VPC in TWC sections (Series B)

Fig. 6(a) compares the variation of axial load with duration of fire for TWC and VPC columns. As the duration of fire is increased from zero to one hour, strength of both type of specimens is found to decrease.

The strength reduction (Fig. 6(b)) in TWC column is found to be higher than that of VPC column for up to a duration of 30 minute. But as duration of fire is increased to one hour, VPC columns showed higher reduction in strength (about 52%) compared to TWC columns (about 44%).

The performance of VPC in-fill in TWC sections is much better than that of VPC columns (Fig. 6(b)). The strength reduction in VPC in-fill column is less than that of VPC columns. The lower reduction of strength in VPC in-fill columns confirms the lesser susceptibility of in-fill concrete to fire in TWC section although overall fire resistance of the sections may be lower.

The weight of both VPC and TWC columns decrease with the increase of duration of fire (Fig. 6(c)). Both of them show similar trend of variation with maximum reduction in weight of about 14% for a duration of one hour.

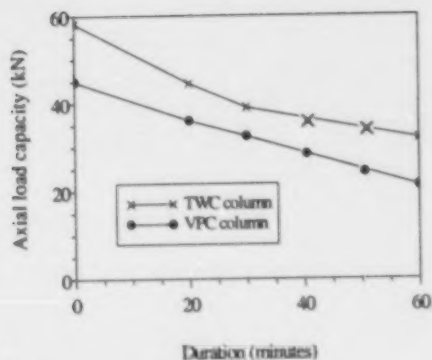


Fig. 6a: Variation of strength with fire duration

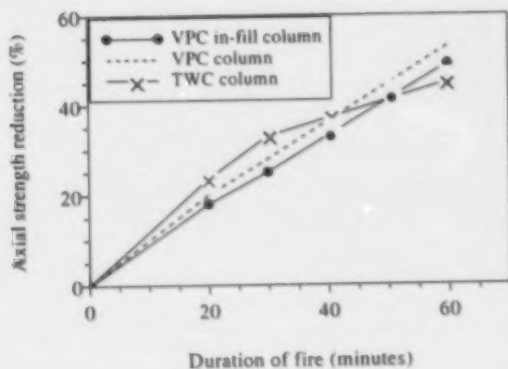


Fig. 6b: Strength reduction as a function fire duration

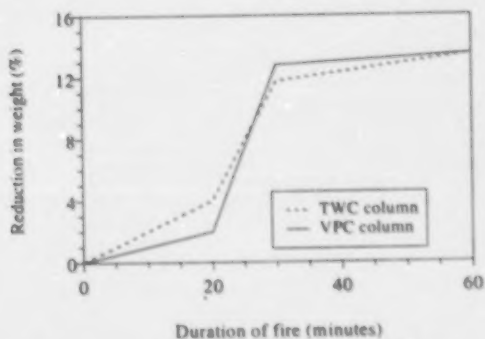


Fig. 6c: Effect of fire duration on the weight of the specimen

4.3 General discussion and ongoing research

It is required that the structural concrete should preserve good fire rating characteristics over a desired length of time. Strength of concrete decreases when exposed to a temperature in excess of 35°C and under conditions allowing loss of moisture content. On the other hand mild steel can lose one-half of the yield strength at about 600°C. The 52% strength loss in the VPC specimens can be attributed to loss of about 14% reduction in weight (may be due to loss of moisture content) and induced high temperature gradients causing hot surface layers tending to separate and spall from cooler interior concrete layer. However, light weight VPC should exhibit more fire resistance characteristics than ordinary aggregate concrete due to lesser tendency to spall and loss of lesser proportion of its original strength with the rise in temperature. The 44% reduction in strength of TWC specimens can be related to combined effect of loss of strength in concrete and steel. Lower thermal conductivity of VPC can be disadvantageous for TWC section as it can induce rapid rise in temperature in steel sheeting.

The resistance of concrete to fire and to elevated temperature depends on many factors such as: duration of fire, temperature, moisture conditions, concrete mix: leaner or richer, aggregate properties etc. More investigation is needed to fully understand the fire resistance of VPC in TWC sections. Investigation is now under progress covering all these parameters: varying duration of fire from 0 to 4 hours, temperature from 30°C-1000°C, moisture conditions from saturated surface dry to air dry, concrete mix from 1:2:3 to 1:2:4 and using ordinary and pumice aggregate. For TWC sections, thickness of steel sheeting will be another variable parameter.

5 Conclusions

The properties of volcanic pumice concrete is presented based on experimental results. Durability of VPC subjected to fire (elevated temperature of 600°C) especially in connection with TWC section is judged based on test observations and strength characteristics. The strength is found to decrease with the increase of duration of fire. The strength reduction in VPC specimens is found to be 52% compared to 44% in TWC sections when subjected to a fire of one hour duration. However, the fire resistance of VPC in TWC section is better than its individual resistance.

6 Acknowledgements

The author is grateful to the Papua New Guinea University of Technology for providing financial assistance in this project. The author is also grateful to the Technical staffs of the materials laboratory and Mr. Lucas Mol of the department of Civil Engineering for assistance.

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15 MICROSTRUCTURE OF AUTOCLAVED AERATED CONCRETE SUBJECTED TO CARBONATION

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Abstract

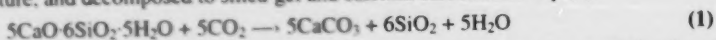
Autoclaved Aerated Concrete(AAC) has been produced more than 30 years in Japan. Therefore, durability and service life of AAC become subjects of urgent study. Microstructure of AAC, which is related to cracking, changed during carbonation. Field AAC panels (Field-AAC) aged 5 to 33 years and laboratory-carbonated AAC blocks (Labo-AAC) were subjected to analysis. Microstructure of the samples was examined with XRD, TG-DTA, EPMA, mercury porosimeter, NMR, BET, and water vapour sorption isotherms and compared with the drying shrinkage data. Changes in peak of silicate structure in ²⁹Si MAS NMR spectrum, approximately 1nm-micropore volume and drying shrinkage during carbonation were different between Field-AAC and Labo-AAC, while changes of crystalline phases (Tobermorite-11Å and calcium carbonate) and total pore volume were not different. It was concluded that the microstructure of silica-gels originated from the decomposed Tobermorite-11Å showed significant differences according to the carbonation conditions.

Keywords: AAC, microstructure, cracking, field sample, laboratory-carbonated sample, drying shrinkage, Tobermorite-11Å, calcium carbonate, durability, carbonation, ²⁹Si MAS NMR, water vapour sorption isotherms,

1 Introduction

Autoclaved aerated concrete (AAC) has been produced in great quantity more than 30 years in Japan. Therefore, durability and service life of AAC become subjects of urgent study. One of the most effective factors for the durability of AAC is

carbonation in which Tobermorite-11Å ($5\text{CaO} \cdot 6\text{SiO}_2 \cdot 5\text{H}_2\text{O}$), the main binding mineral of AAC, reacts with atmospheric carbon dioxide gas in the presence of moisture, and decomposed to silica-gel and calcium carbonate as eq. (1).



Carbonation of the ordinary concrete, which is the reaction of CSH-gel and calcium hydroxide with carbon dioxide gas, leads to the loss of alkalinity and the rust resistance. That is why the neutralization is the major subject for carbonation of the ordinary concrete.

On the other hand, the main binding mineral of AAC is Tobermorite-11Å which is originally neutral because AAC is cured in the autoclave at 180°C and 10 atms sufficiently, regardless of several raw materials, such as quartz sand, cement, calcium oxide, slag and disposed AAC powders. The reinforced steel in AAC is coated with rust-resistant materials previously because Tobermorite-11Å doesn't have an effect of that. Therefore, the subject for carbonation of AAC is not the neutralization but the changes of matrix itself. During carbonation, it is occurred that not only changes of minerals but also changes of microstructure which leads to the degradation such as decrease of strength and, especially, growth of lattice-like cracking. That is why so many studies have been made for the changes of microstructure during carbonation.

(Sauman 1971, 1972) studied the changes of compressive strength, carbonation shrinkage and the 'pseudomorphism' shape-changes of Tobermorite-11Å crystals which showed that only the marginal parts of the crystals were somewhat frayed and unsharpened, under the accelerated conditions of 1-30 vol.% CO_2 and 50-100% R.H. (Sun et al. 1985) observed the 3 types of AAC samples made of different raw materials, by means of SEM after carbonation under the accelerated conditions of 50 vol.% CO_2 and 50 % R.H. They suggested that calcium carbonate deposited on the walls of air pores, silica-gel maintained the original network structure of Tobermorite-11Å, and these changes were not different among samples. (Iwasaki and Tada 1985) showed the changes of pore volume proportions with distinctions of submicro-, meso- and macro-pores by means of water vapour sorption isotherms and mercury porosimeter for the accelerated carbonated AAC under conditions of 20 vol.% CO_2 and 100 % R.H. (Ikeda et al. 1991) indicated that the silicate structures in Tobermorite-11Å were changed from double-chain structures to 3-dimensional framework structures by means of ^{29}Si MAS NMR spectrum during carbonation under accelerated conditions of 10 vol.% CO_2 and 60 % R.H. However, these studies were all investigated for the accelerated carbonated AAC without correlating with the carbonation under practical conditions.

Several studies about the carbonation under practical conditions were investigated. (Hanecka et al. 1997) showed the increase of the specific gravity. (Dapkus and Stankevicius 1985) showed the increase of the amount of water content. However, the microstructure changes of AAC under practical conditions, which is extremely important to examine the durability and service life, have not been investigated yet. In addition, it is very important to make clear the difference of the microstructure changes between under practical conditions and accelerated conditions

for prediction of degradation and remaining years by means of accelerated carbonation methods.

This study indicated the microstructure changes of AAC under practical conditions and accelerated carbonated conditions with reference to the approach of structural levels proposed by (Wittmann 1985).

2 Experimental

2.1 Samples

Field AAC panels (Field-AAC) aged 5 to 33 years under practical conditions, laboratory-carbonated AAC blocks (Labo-AAC) under accelerated conditions of 3 vol.% CO₂, 90% R.H. and 20°C, and non-treated AAC blocks (Nontreated-AAC) were subjected to analysis. Field-AAC were removed from several buildings in Japan. The usual climate in Japan is 0.03 vol.% CO₂, 25 to 95% R.H. and 0 to 35°C. Ordinary AAC blocks made by Siporex Yokohama factory of Sumitomo Metal Mining Co., Ltd were used for Labo-AAC and Nontreated-AAC. They were shaped in 80*40*10mm for several analysis and 40*40*160mm for measuring length changes.

2.2 Measurement of the degree of carbonation

The amount of combined carbon dioxide and calcium oxide were measured to determine the degree of carbonation. TG-DTA was used for analysis of the amount of combined carbon dioxide excluding the amount of adsorbed carbon dioxide gas and carbonated salt other than calcium carbonate. The amount of combined carbon dioxide were measured as the weight loss from 600 to 800°C corresponding to the decomposition of calcium carbonate. The amount of calcium oxide was analysed by ICP. The degree of carbonation (Dc) can be given by

$$Dc (\%) = [(C - C_0) / (C_{max} - C_0) \times 100], \quad (2)$$

where C, C₀ and C_{max} are the amount of combined carbon dioxide for each samples, that of Nontreated-AAC and that when all calcium oxide become calcium carbonate.

2.3 Analysis of crystalline phases and microstructure

Crystalline phases were analysed by XRD. Microstructure was analysed by mercury porosimeter, EPMA, SEM, BET specific surface area analyzer, water vapour sorption isotherms and ²⁹Si MAS NMR.

2.4 Measurement of drying shrinkage

Drying shrinkage was measured as the length change from the saturated water condition to the equilibrium state under 45% R.H.

3 Results and discussion

3.1 Degree of carbonation (Dc)

Changes of Dc for Field-AAC and Labo-AAC as a function of time are shown in Fig.1(a) and (b), respectively. For Field-AAC, Dc increased with exposed years and saturated to 60% approximately after 30 years. For labo-AAC, Dc increased with treated days and saturated to 60% approximately after 40 days. In the following discussion, Dc will be used as the basis for estimation because the changes of Dc were very similar between Field-AAC and Labo-AAC.

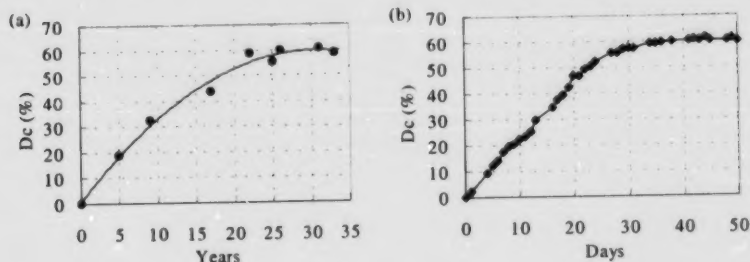


Fig.1: Dc for (a) Field-AAC and (b) Labo-AAC

3.2 Changes of crystalline phases

Changes of crystalline phases of Field-AAC and Labo-AAC analysed by XRD are shown in Fig.2(a) and (b), respectively. In both samples, changes of crystalline phases are similar. With increase of Dc, Tobermorite-11Å decreased and calcite increased. When Dc was 30%, vaterite began to increase. Therefore, the decomposition of Tobermorite-11Å and the deposition of calcium carbonate were similar to each other though carbonation rate was different extremely because of the differences of atmospheric carbon dioxide concentration and relative humidity.

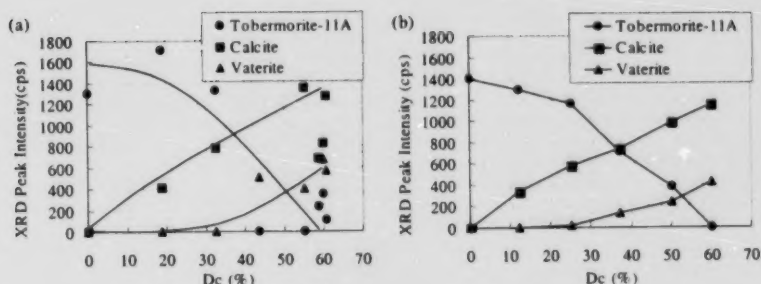


Fig.2: XRD peak intensity for (a) Field-AAC and (b) Labo-AAC

3.3 Changes of microstructures in levels of 10nm to μm

Pore size distributions by mercury porosimeter of Nontreated-AAC, Field-AAC ($D_c=60\%$) and Labo-AAC ($D_c=60\%$) are shown in Fig.3. The total pore volume of Field-AAC and Labo-AAC decreased from 0.8ml/g to 0.6ml/g by carbonation, particularly in the typical pores of AAC around 10 to 1000nm corresponding to interparticle pores of Tobermorite-11Å (Prim and Wittmann 1983).

Ca and Si distributions for Nontreated-AAC, Field-AAC ($D_c=60\%$) and Labo-AAC ($D_c=60\%$) by EPMA cross-section analysis are shown in Fig.4. For both Field-AAC and Labo-AAC, the distribution of Ca atoms deposited locally in the matrix during carbonation, though the distribution of Si atoms unchanged. It was observed that Ca ions migrated from Tobermorite-11Å and reacted with carbon dioxide gas, though Si ions remained.

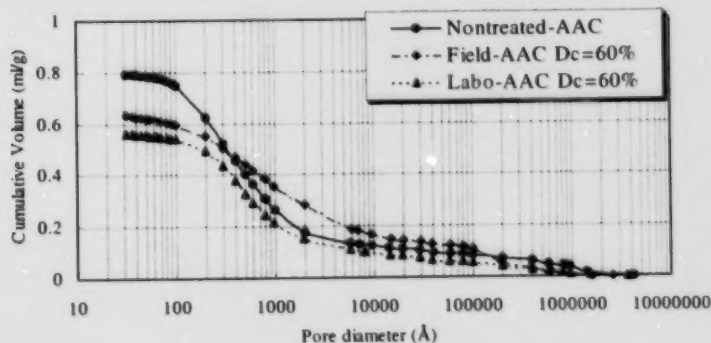


Fig.3: Pore size distribution by mercury porosimeter

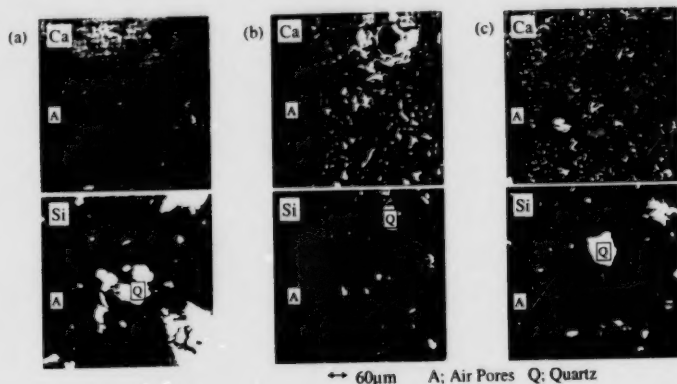


Fig.4: Ca and Si distributions for (a) Nontreated-AAC, (b) Field-AAC (Dc=60%) and (c) Labo-AAC (Dc=60%) by EPMA cross-section analysis

SEM micrographs in air pores for Nontreated-AAC, Field-AAC (Dc=60%) and Labo-AAC (Dc=60%) are shown in Fig.5. The shapes of Tobermorite-11Å particles for Nontreated-AAC were planer. For carbonated samples, the shapes of silica-gels, which is called silica-skeleton, originated from decomposed Tobermorite-11Å were different slightly. For Labo-AAC, only the marginal parts of the silica-gels were somewhat frayed and unsharpened as shown in the preceding studies (Sauman 1972). On the other hand for Field-AAC, the shapes of silica-gels were similar to those of Tobermorite-11Å of Nontreated-AAC. Anyway, the planer particle shapes and the interparticle pores by geometrical arrangement of those were not changed significantly during carbonation under both practical and accelerated conditions. Therefore, it was clear that the decrease of pore volume during carbonation was caused by the deposition of calcium carbonate into the interparticle pores around 10 to 1000nm.

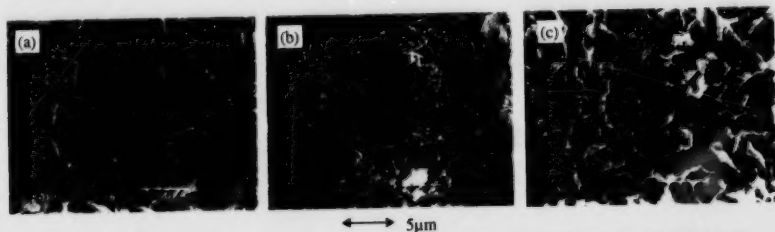


Fig.5: SEM micrographs in air pores for (a) Nontreated-AAC, (b) Field-AAC (Dc=60%) and (c) Labo-AAC (Dc=60%)

3.4 Changes of microstructures in levels of atomic arrangement to 10nm

The changes of BET specific surface area are shown in Fig.6. For Labo-AAC BET specific surface area increased from 20 to 60 m²/g during carbonation. For Field-AAC, however, it was decreased to 10 m²/g. The pore size distribution by BET specific surface area is shown in Fig.7. Micropores approximately 1 and 6 nm can be observed for Nontreated-AAC. All ranges of micropores decreased for Field-AAC. Micropore volume of approximately 1 nm increased extremely for Labo-AAC. It was observed that this differences of pore structures around nanometer levels, which had great influence to BET specific surface area, indicated the differences of structures in silica-gels between Field-AAC and Labo-AAC because crystallines of calcium carbonate were large enough to be detected by XRD.

Water vapour sorption isotherms for Nontreated-AAC, Field-AAC (Dc = 60%) and Labo-AAC (Dc = 60%) are shown in Fig.8. It was recognised that there was hysteresis loop only for Field-AAC. This was type B hysteresis loop classified by de Boer indicating pore structures of slit or parallel plane. It was observed that silica-gels of Field-AAC had layered structures in which adsorption and desorption of water were possible. Spectrum of ²⁹Si MAS NMR for Nontreated-AAC, Field-AAC (Dc = 60%) and Labo-AAC (Dc = 60%) are shown in Fig.9. Chemical shifts of ²⁹Si MAS NMR are shown in Table 1. For Nontreated-AAC, Q² and Q³ spectrum which indicated the silicate double-chain structures were found as reported by (Wieker et al. 1982). For Labo-AAC, Q⁴ spectrum which indicated the silicate 3-dimentional framework structures similar to ordinary silica-gels were found as reported by (Ikeda et al. 1991). For Field-AAC, however, Q³ spectrum, which indicated the silicate sheet-like structures and had not been reported yet, were found.

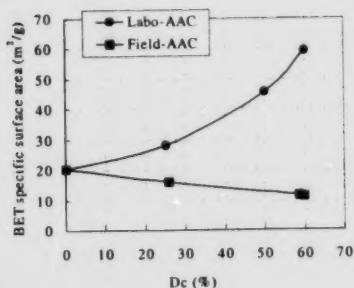


Fig.6: BET specific surface area

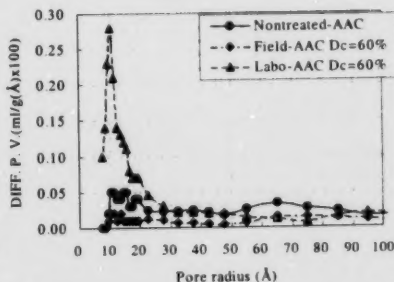


Fig.7: Pore size distribution by BET

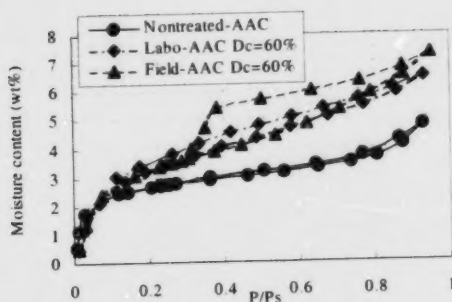


Fig.8: Water vapour sorption isotherms

Tobermorite-11Å has layered structures in which silicate double-chain structures are sandwiched between CaO layers. For Field-AAC, carbonation reaction was relatively slow and mild because carbon dioxide gas concentration was as low as 0.03 vol.% in the air. Therefore, it was observed that the silicate structures were rearranged slightly from double-chain to sheet-like structures and the crystalline became bigger just like Ostwald ripening during carbonation under practical condition. It was also observed for Field-AAC that the layered structures of silica-gel was the result of the silicate sheet-like structures and the planer particle shapes of Tobermorite-11Å were not changed because of the moderate rearrangement of silicate structures.

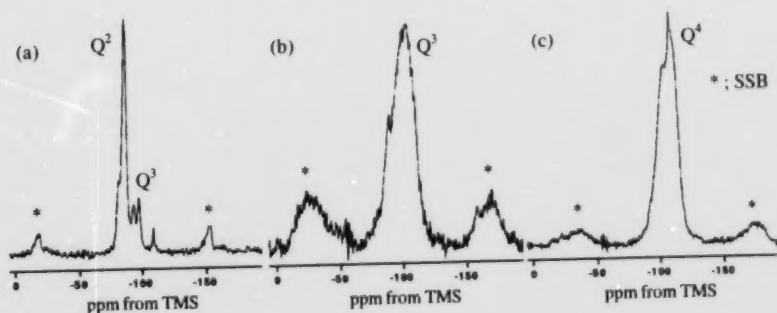


Fig. 9: ^{29}Si MAS NMR spectrum for (a) Nontreated-AAC, (b) Fields-AAC (Dc=60%) and (c) Labo-AAC (Dc=60%)

For Labo-AAC, on the other hand, ambient carbon dioxide gas concentration was as high as 3 vol.% in the atmosphere, so that carbonation reaction was very rapid and hard. Therefore, it was observed for Labo-AAC that the silicate structures were rearranged extremely from double-chain to 3-dimensional framework structures and the crystalline were divided smaller resulting in the increases of BET specific surface area and approximately 1 nm-micropore volume. It was also observed for Labo-AAC that the marginal parts of planer particle shapes of Tobermorite-11Å were altered to somewhat frayed and unsharpened because of the considerable rearrangement of silicate structures.

Table 1: Chemical shifts of ^{29}Si MAS NMR

ppm from TMS	Q ²		Q ³		Q ⁴		
	1Al	0Al	1Al	0Al	1Al	0Al*	0Al
Non-treated AAC	-80.3 (28)	-85.1 (100)	-91.9 (16)	-96.1 (17)		-107 (4)	
Field-AAC (Dc=60%)		-87.3 (29)		-99.3 (100)			
Labo-AAC (Dc=60%)					-103 (100)	-107 (5)	-110 (68)

() : Relative peak intensity

* : Quartz

3.5 Drying shrinkage

Drying shrinkage as a function of Dc is shown in Fig.10. Drying shrinkage increased with the increase of Dc from 0.03% to 0.06% and 0.11% for Field-AAC and Labo-AAC, respectively. Drying shrinkage of AAC is the result of that of Tobermorite-11Å or silica-gels which are the main binding minerals for Non-treated and carbonated AAC. Therefore, it was observed that the increase of drying shrinkage was mainly due to the change of main binding minerals from Tobermorite-11Å to silica-gels. It was also observed that changes of silica-gel structure in Labo-AAC were more excessive than that in Field-AAC so that drying shrinkage for Labo-AAC was larger than that for Field-AAC. In other words, behaviour of macro-level such as drying shrinkage was influenced by microstructure such as atomic arrangement or crystalline structures.

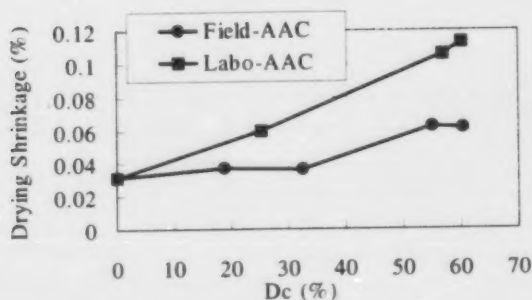


Fig. 10: Drying shrinkage as a function of Dc

4 Conclusions

Microstructure changes of AAC were investigated during carbonation under practical conditions, Field-AAC and accelerated conditions, Labo-AAC.

Changes of crystalline phases and microstructures of 10nm to μm were similar between Labo-AAC and Field-AAC as follows;

- (1) Dc increased with time and saturated at 60%.
- (2) With increase of Dc, Tobermorite-11Å decreased and calcite increased. After Dc reached 30%, vaterite increased.
- (3) The pore volume decreased and the Ca atoms were deposited locally in the matrix.
- (4) The planer particle shape and the interparticle pores were not changed significantly.

Changes of microstructures of atomic arrangement to 10nm and drying shrinkage were different between Labo-AAC and Field-AAC as follows;

- (5) BET specific surface area increased from 20 to 60 m^2/g for Labo-AAC, but decreased to 10 m^2/g for Field-AAC. Micropore volume of approximately 1 nm increased extremely for Labo-AAC.
- (6) Hysteresis loop indicating the layered structures on the water vapour sorption isotherms were observed only in Field-AAC.
- (7) In the ^{29}Si MAS NMR spectrum, Q^4 spectrum indicating the silicate 3-dimensional framework structures were found for Labo-AAC, however, Q^3 spectrum indicating the silicate sheet-like structures were found for Field-AAC.
- (8) Drying shrinkage increased with increase of Dc from 0.03% to 0.06% and 0.11% for Field-AAC and Labo-AAC, respectively.

5 Acknowledgments

The authors are grateful to Prof. K. Okada and Mr. Y. Kameshima of Tokyo Institute of Technology, for their support in the measurement of water vapour sorption isotherms.

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16 DURABILITY OF MORTAR LININGS IN DUCTILE IRON PIPES

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Abstract

Mortar linings in ductile iron pipes for distribution of water have been investigated with respect to micro-structure, composition, durability and leach of chemical components. Scanning Electron Microscopy (SEM) and Wave Dispersive Analysis of X-ray (WDAX) have been used to study degradation of mortar linings which have been in service for 10 - 20 years. The same techniques have also been used to follow degradations and leaching of new mortar linings exposed to aggressive solutions in laboratory. Relatively good correlation has been found between the laboratory tests and the degradation of linings of pipes in service. Reliable methods for examination and testing of mortar linings in new pipes will definitely be a valuable implement for municipal water works to specify a wanted composition and quality of mortar linings.

Keywords: water pipes, mortar linings, type of cement, durability

1 Introduction

Among our building materials few of them affect our daily life as much as the water pipe line network. A steady delivery of water without any interruptions is of great importance to our quality of life. Also from an economic point of view it is important to have pipes of good quality in order to secure a long service life of the pipes without considerable expenses of maintenance. Traditionally, water mains consisted of ductile iron pipes without any protection against corrosion. From about 1970 - 80 internally mortar lined iron pipes came into use in Norway. Due to a high pH the mortar linings are supposed to protect the iron pipes against corrosion. However, dependent on physical and chemical properties of the mortar and / or the

quality of the water, any mortar lining is to be more or less attacked by the streaming water, and chemical compounds like calcium and alkalies are leached from the mortar linings. Unfortunately, the greater part of the water supply in our country comes from surface water. Drinking water has a low pH (average: 6.7), a low hardness (average: $\text{Ca}^{2+} = 2.6 \text{ mg/l}$), and a low alkalinity (average: 0.05 - 0.1 mmol/l). Most of the local water works are small and cannot afford to invest in water treatment equipment to increase pH and mineral content by adding lime or alkalies. Often the results are a comparatively short "service life" of mortar linings, which means great expences on maintenance, repairs, and replacements of pipes, and bad water-quality due to leach of chemical components from the mortar.

2 Degradation mechanisms

The capacity of mortar and concrete to withstand chemical degradation can be related to three of the material's characteristics:

- Permeability
- Alkalinity
- Reactivity

Permeability, the capacity against diffusion of fluids and gases into the materials, is controlled by following parameters:

- Water / Cement ratio
- Condition of hardening
- Additives as silica fume, blast furnace slag, and fly ash.

However, even if all precautions have been taken to produce a dense concrete surface, fluids and gases will to a certain degree penetrate into a construction.

Chemical degradation of mortar and concrete due to penetration of detrimental solution into the material may in according with the reaction mechanisms be divided into following categories:

- Demolition of cement paste caused by leaching, acid attacks, and exchanges of ions.
- Growth of expanding substances caused by interaction between cement minerals or aggregates and penetrated salts, as examples, sulphate attacks and alkali aggregate reactions.

Even if the quality and the design of the mortar lining are good, there will always exist a deleterious effect on cement minerals when mortar linings are exposed to streaming water. Degree and rate of degradation will, besides mortar quality, depend on the natural properties, quantity, and streaming velocity of the water.

If the leaching of the cement paste is not stopped on the surface, but continues

deeper into the mortar lining, the mortar may lose its capacity to protect the iron pipe against corrosion.

3 Mortar Linings

3.1 Methods of lining

Commonly two methods are used for lining of ductile iron pipes:

- Centrifugal method. The mortar mix is cast on the iron pipe when it is under a very high rotation velocity. This gives a cement mortar lining with a very cement rich layer towards the water with a porous and sand rich layer towards the iron pipe. A scanning microscope photo of the micro structure of this lining is shown in Figure 1.



Fig. 1: Centrifugal lining method

- Centrifugal projection head method. By this method the mortar mix is cast on a still-lying iron pipe through a spinning head. This gives a cement mortar lining with an equal distribution of cement paste, pores and sand throughout the whole thickness of the lining. A scanning microscope photo of the micro structure of this lining is shown in Figure 2.

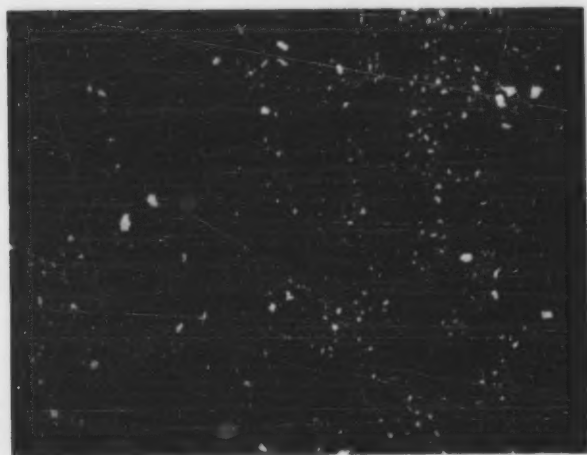


Fig. 2: Centrifugal projection head method

3.2 Cements

If the mortar lining is exposed to soft water with low pH and low content of minerals, the type of cement used in the mortar may have decisive consequences for the durability of the lining. According to DIN 2614 following cements are allowed for mortar linings in pipes transporting drinking water, Ordinary Portland Cement (OPC), High Alumina Cement (HA cement), Blast Furnace Slag Cement (BFS cement), Fly Ash Cement (FA cement), and Sulphate Resistant Portland Cement (SR cement). One of the main components of a hardened OPC paste is calcium hydroxide [$\text{Ca}(\text{OH})_2$], and this is also the "weak" mineral of a cement paste and is easily leached from a cement mortar. Among the cements mentioned, OPC and SR cement have the larger contents of calcium hydroxide. Due to the raw materials HA cement is relatively low in $\text{Ca}(\text{OH})_2$. Also mortars based on BFS cement and FA cement have reduced content of $\text{Ca}(\text{OH})_2$ partly due to the lower content of OPC in these cements and partly because of the pozzolan reaction between calcium hydroxide and the mineral substitute in the cements.

4 Aims

The aim of the project was to find which of the mortar linings is the most suitable one for a specific water quality. This should be done by, firstly, to carry out a condition survey of mortar linings taken from existing pipes lines at different parts of the country. Secondly, to use accelerated laboratory test methods to investigate the durability of specimens taken from new linings, and to examine the conformity between the laboratory tests and the actual deterioration caused in performance by streaming water.

5 Experimental details

5.1 Materials

The experiments included mortar linings based on either Ordinary Portland Cement, High Alumina Cement, Blast Furnace Slag Cement, or Fly Ash Cement.

5.2 Test methods

5.2.1 Scanning electron microscope (SEM)

The mortar linings have been investigated by use of Scanning Electron Microscope (SEM). Using Back Scattered Electron modus (BSE) in Scanning Electron Microscope (SEM) for examination of mortar gives the possibility to distinguish between different phases and their distribution. It is important that the sample is properly plane polished. The back scatter process depends strongly on the average atom number of the sample, sample density and the energy of the primary electrons. Phases with high average atom number, as iron (Fe), and / or samples with a dense structure will have bright colours on a BSE image. On the other hand, phases with low average atom number, as magnesium, and samples with porous structures will appear dark.

5.2.2 Wave dispersive analysis of X-rays (WDAX).

Together with a SEM-investigation Wave Dispersive Analysis of X-rays (WDAX) diagrams have been recorded. WDAX gives semiquantitative compositions of solids. The principle for the method is based upon the exitement of the electrons of each atom in the sample. When the electrons return to their stable energy level, the atoms in the sample will emit X-rays of a certain energy specific for each chemical element. The results will appear as peaks in a diagram with variation of energy as X-axis. The position (X) determines the element, while the height (Y) of the peaks gives the amount of the specific elements in the sample. By use of WDAX small volumes ($\approx 1 \mu\text{m}^3$) of a sample are analysed with respect to chemical composition.

5.2.3 Accelerated Leaching Test

An accelerated leaching test has been performed by using a modification of the Dutch method NEN 7345. According to this method samples of concrete or mortar are exposed to deionized water or acid solution for 64 days. At certain intervals during this period samples are taken from the solutions and analysed with respect to calcium and aluminium.

5.3 Experiments

5.3.1 Mortar linings in used pipes

Specimens of mortar linings were impregnated with epoxy and plane polished. Deterioration of the mortars has been studied by use of scanning electron microscope (SEM) and wave dispersive analysis of x-rays (WDAX). In addition leaching of elements (CaO and Al_2O_3) from the matrix has been determined by analysing the

cement pastes at certain distances from the water side towards the side of the iron pipe.

5.3.2 Mortar linings in new pipes

For accelerated deterioration tests of mortar linings specimens of about 130 x 70 mm were cut from the pipes, isolated by epoxy, and exposed to acetic acid buffer solution (pH = 4.6) for 64 days. The solution was kept in movement throughout the testing period. A buffer solution was chosen for the experiments because the buffer solution gives a relatively permanent pH even when $\text{Ca}(\text{OH})_2$ is leached from the mortar. This gives allowance to compare a buffer solution with streaming water. During the exposure period samples have been taken from the solutions and analysed with respect to contents of calcium (Ca^{2+}) and aluminium (Al^{3+}) by use of atomic absorption spectrophotometer. Durability of the mortar linings has been estimated by SEM / WDAX examinations of the specimens before and after exposure. In addition the thickness of the linings has been measured before and after exposure in the buffer solution.

6 Results and discussion

6.1 Mortar linings from pipes taken from networks

The investigation included 15 mortar lined iron pipes taken from pipelines at several water works throughout the country. The pipes had been in use for 10 - 20 years. SEM (BSE) and WDAX showed following results:

- The cements used in the mortars were Ordinary Portland, Blast Furnace Slag Cement or Fly Ash Cement.
- Both the centrifugal method and the centrifugal projection head method had been used in lining the pipes.

All the specimens showed degradation of the mortar lining in form of leached elements. This means that water delivered to the people (consumers) may not have the wanted quality due to a high pH caused by leaching of calcium and hydroxyl ions from the mortars. At the same time the leaching of calcium hydroxide also causes a gradually loss of capacity to protect the iron pipe against corrosion. A typical SEM photo of a mortar lining taken from a network is shown in Figure 3. Three distinct layers can be seen on the photo. That means a gradually leaching of the cement paste. The black area at the bottom of the photo, which shows the "water side" of the mortar, indicates a strongly leached paste, whilst the inner white area indicates that no leaching of elements has occurred.

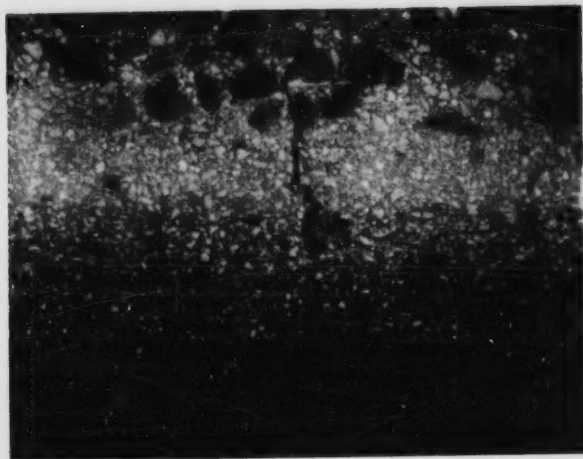


Fig. 3: Leached mortar linings

A considerable reduction of the linings' thickness could be observed in some of the pipes, especially in those lined by the centrifugal projection head method.

Linings from a couple of pipes which had been exposed for about 20 years to water quality: pH = 6.5, calcium (Ca^{2+}) = 1,5 mg / l, alkalinity = 0.04 mmol / l, were totally destroyed, and by SEM / WDAX iron rich layers could be observed in the remaining mortar lining. This result indicates that corrosion of the iron pipes has started.

Due to these results, which showed that mortar linings could be in a rather bad condition after 10 - 20 years in use, it was decided to do some laboratory tests on new mortar linings in order to evaluate which type of mortar i. e. cement and method of lining were the most durable one.

6.2 Mortar linings in new pipes

The investigation included mortar linings based on the following types of cement, FA cement, BFS cement, and HA cement. The mortar based on FA cement had been cast on to the iron pipe by the centrifugal projection head method. Whilst the centrifugal method had been used in lining the pipes with the BFS cement and HA cement mortars.

Measurements of the linings' thickness before and after exposure in acetic acid buffer solution showed:

- Non reduction in thickness of the mortar based on HA cement.
- A reduction of 0.5 - 1.0 mm of the mortar based on BFS cement.
- A reduction of about 3 mm of the mortar based on FA cement.

6.2.1 Accelerated leaching test

The total quantities of calcium and aluminium leached from the mortars during 64 days are given in table 1. The results are calculated as oxides, and given in mg / cm² exposed mortar surface.

Table 1: Leached CaO and Al₂O₃ (mg / cm²) from mortar linings after 64 days of exposure in acetic acid buffer solution.

Component	Type of cement		
	HA cement	FA cement	BFS cement)
CaO (mg/cm ²)	39.9	42.7	36.7
Al ₂ O ₃ (mg/cm ²)	26.2	0.65	0.56

The relatively large quantity of Al₂O₃ leached from the HA cement based mortar, is due to the raw material used in the production of this cement. When following the quantities of leached Ca and Al from the mortars at each testing time, the results indicated a very rapid leaching from the mortars based on FA cement and BFS cement, respectively. The leaching from the HA cement based mortar, however, went on through a longer period of time.

6.2.2 SEM / WDAX

SEM / WDAX examinations showed leached linings into a depth of 0.75 mm for the HA cement based mortar. Corresponding results were 1.5 mm and 2.0 mm respectively for the FA cement and the BFS cement based mortars. It should be noticed, however, the great reduction of thickness of the FA cement based lining during the test period. A WDAX diagram with respect to leached Ca and Al from the HA cement mortar is shown on Figure 4.

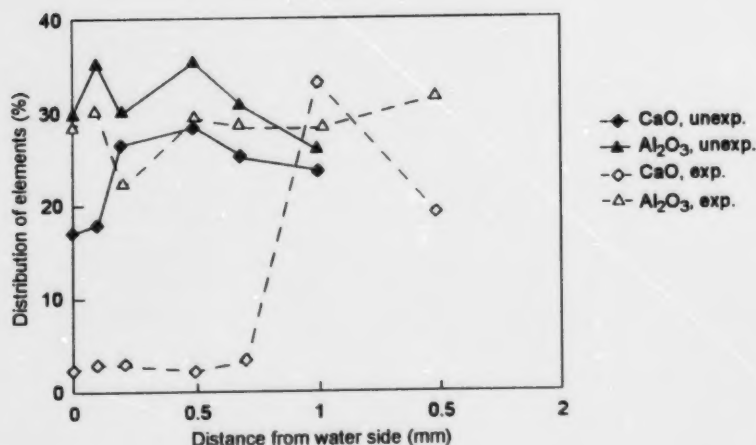


Fig. 1: High Alumina mortar. Content of CaO and Al₂O₃ before- and after exposure

7 Conclusion

The results from the accelerated test indicated following:

- When exposed to concrete aggressive solution, mortars cast on to the iron pipe by the centrifugal projection head method seems to lose material to a larger degree than mortars cast on to the iron pipe by the centrifugal method. This has also been observed in examinations of used pipes taken from networks. This means that the dense cement layer obtained by the centrifugal method, has a very small permeability.
- Mortar based on High Alumina cement is more durable in concrete aggressive solutions with low pH compared to the other mortars included in the experiment. This means that a High Alumina cement based mortar lining will have a longer "service life" at low pH than linings based on Blast Furnace Slag cement or Fly Ash cement.
- The smallest total amounts of calcium and aluminium leached into the solution were measured from the Blast Furnace Slag cement.
- Rather unexpected high amounts of calcium and aluminium were leached from High Alumina cement mortar.

8 Recommendation

Based on the results from the accelerated test programme, ductile iron pipes lined by the centrifugal method with mortar based on Blast Furnace Slag cement, can be recommended for pipelines, which are going to be exposed to a moderate water quality.

For extreme water qualities, water with low pH, low content of calcium and low alkalinity, High-Alumina cement based mortar linings should be recommended.

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17 EXPERIMENTATION ON PHYSICAL DURABILITY OF MULTILAYER PANELS CAST IN SITU

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Abstract

This paper reports of durability experimental assessment of multilayer concrete panels cast in situ (sandwich type). The panels consist of a steel reinforcement positioned on the layer of insulating material on both sides. Once installed in situ, the panels are completed with several layers of shotcrete.

This study included accelerated tests of cyclic action of atmospheric agents (T, RH) on panel-samples made with different concrete mixes. Phenomena of fissure and decohesion between layers were analysed both in short term (accelerated fissure test) and in long term (accelerated ageing test). Accelerated ageing test was performed in a climatic chamber designed and built to house on two sides the samples.

The tests were carried out along with a visual and instrumental investigation of the product decay. The evolution of the *superficial state* was measured reading the fissures length and width, while the evolution of the *adhesion state* between layers was investigated by means of diagnostic non destructive techniques such as Infrared termography and Laser-Doppler vibrometry, recently used in building.

Keywords: accelerated tests, climatic chamber, Infrared termography, Laser-Doppler vibrometry, multilayer panels, physical durability, shotcrete

1 Introduction

Subject of this report is durability experimental tests carried out on two types of multilayer reinforced concrete panels (sandwich type) *cast in situ*.

Initially the panels are shaped in plant as a spacial reticular trellis (steel reinforcement) positioned on a central layer of insulating material (polyurethane or polystyrene). Only once installed in situ, several layers of shotcrete are applied on both sides of the panels, operating as sandwich panels. It is possible to produce both bearing walls (structural) and infill walls (non structural).

The types tested come from patents belonging to two Italian firms, the Rein (Ascoli Piceno) and the Monolite (Fano). In the first case a structural wall (R445S - 16 cm) was chosen, in the second case a non structural type (PSTG - 14 cm) (Fig. 1).

None of the mentioned firms has carried out a specific study in terms of durability. This problem is real, in consideration of the corrosion of the reinforcement (reticular trellis), covered by thin layers of concrete, with the risk of cover ejection.

However, it does not seem that the buildings constructed so far have suffered problems of this kind, but this is a recent technology, while the average lifetime of a building is allowed for decades, at least in Europe. As a preventive measure, it can thus be important to control the superficial fissure state, responsible for the penetration of agents deleterious for service life, and how this evolves in the time.

What's more, there are no available data about the adhesion state between insulating material-concrete and/or concrete-concrete. In particular the latter aspect is related with the shotcreting technology and with the need to repeat the operation at following time intervals to reach the thickness required on both sides of the panel.

A bad execution in situ of this technology may cause decohesion phenomena between layers with peelings, moreover intensified by dimensional variations due to the natural actions.

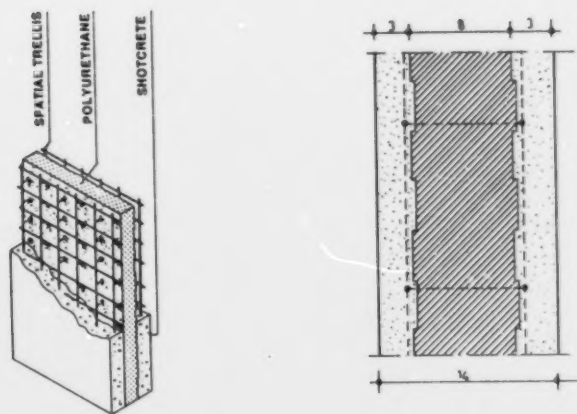


Fig. 1: Panel materials and vertical section (Monolite PSTG type)

2 Research methodology

The research analysed the degradation mechanisms caused by climatic thermohygrometric excursions. The study was based on accelerated tests of cyclic action of atmospheric agents (temperature, relative humidity).

In an initial phase buildings constructed with these systems were investigated in situ to acquire information about the preservation state of the panels. This phase advised the research to a simulation testing of the climatic actions.

The experimentation phase began with a qualitative study of the concrete conglomerate, usually cast in situ (standard), and with the proposal of alternative types of conglomerate mixes (optimized). With regard to shotcreting technology, the conglomerate is consisting of a low granulometry aggregate variously combined only with fine and coarse sand, having a diameter $4 + 5$ mm as a maximum.

Samples of both panels were prepared and two accelerated tests (fissure - ageing) were carried out to study the decay evolution respectively in short term and long term.

3 Experimentation plan

The experimentation was conducted on small size panel samples (30 x 60 cm), produced in laboratory by designing of 4 shotcrete types. For each shotcrete and with regard to each system (Rein and Monolite) 4 samples to test were produced (4 series of samples for each system).

Mixes were constituted by 3 shotcretes variously additived and optimized according to European regulation about mix-design (ENV 206 - 1992), recently adopted in Italy, and by a fourth traditional usually used by the panel producers:

1. Plastic fibre shotcrete (polypropylene) (optimized)
2. Silica fume shotcrete (optimized)
3. Expansive agent shotcrete (optimized)
4. Traditional shotcrete (standard)

Additional elements were used both to oppose fissures affecting panels cast in situ and to improve the rheological properties of the mix for a better layers adhesion.

But the study of the 3 shotcretes proposed showed a lack of specific criteria for the mix-design of conglomerates with small diameter aggregates ($D_{max} < 8$ mm).

In order to remedy this lack, foreign regulations adopted in USA (such as ACI) for studying the optimal granulometry distribution of fine aggregates in the use of pumped and cast concrete were adopted (Fig. 2).

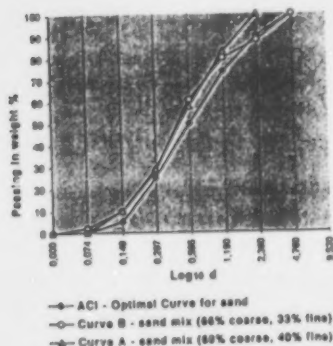


Fig. 2: ACI sand curve and the sand mixes adopted

Two accelerated tests were carried out to study the physical decay of the panels. Aim of the tests was both to reproduce the decay mechanisms affecting the multilayer panels cast in situ and to control the decay evolution produced by the cyclic action of natural agents (such as temperature and humidity).

1. **Accelerated fissure test** (for short-term decay)
2. **Accelerated ageing test** (for long-term decay – thermal cycles)

The *accelerated fissure test* was performed in laboratory simulating conglomerate setting conditions in a hot and sufficiently dry atmosphere.

A set of samples for each system (REIN, Monolite) was submitted to a cycle of hot air forced ventilation for about 10÷12 hours since the casting operation. The thermohygrometric conditions of the testing room were monitored and recorded during the whole time. For promoting the fissure phenomena caused by a fast wet conglomerate drying (plastic shrinkage), specific measures concerning the formwork setup were adopted, such as inserting thread bars, anchored and integral with the formwork. This expedient was selected to encourage the fissures triggered by plastic shrinkage in consequence of contraction. The bars anchorage on the formwork was to prevent that the contraction could produce only a concrete detachment from formwork edge, without the forming of fissures on the surface. The test was aimed at:

- simulating a bad seasoning operation (which often occurs in situ on uniformwork structures) leading to a fast drying process of fresh conglomerate and plastic shrinkage fissures, observed in panels cast in situ; failing a correct wet seasoning, the fast water evaporation is the cause of reticular fissures typical on large structures cast in situ as walls or floorings;
- generating the appearance of a generic microfissure state in short time, so to be a comparison element to observe the decay evolution following up the accelerated ageing test with thermal cycles.

- The *accelerated ageing test* was directed towards to investigate the durability of multilayer panels submitted to continuous dimensional variations due to climatic excursion. The panels tested in the precedent test were submitted to thermal stresses by repeating a definite number of cycles simulating seasonal excursions typical of central Italy climate.

Said test lasted 5 months and the cycles were about 300. The thermal cycle lasts 12 hours and is comprised of 5 phases (3 hot, 2 cold) producing a temperature rise until $T \approx 60^\circ\text{C}$, a maintenance at high relative humidity ($RH \approx 85\%$), and a cooling process with 2 consecutive jumps. These jumps are comprised of an external cold air change phase and of a following damp cooling phase ($T \approx 3^\circ\text{C}$, $RH \approx 95\%$) (Table 1).

The test was performed in a *climatic chamber* capable of effecting temperature and relative humidity variations cycles (Fig. 4).

The samples, subdivided in 2 series (one for system), each one of five samples, were placed on the two lateral sides of the climatic chamber so to repeat on both faces sides the same climatic conditions as those revealing on an exterior panel in situ, but in inverted positions. This means that the internal side, submitted to thermal stresses, corresponds to the external wall of the panel in situ, while the face external to the chamber and exposed to the lab environment ($T \approx 22^\circ\text{C}$, $UR \approx 55\%$) corresponds to the similar internal wall of an inhabited room (Fig. 3).

Said test was for a double reason:

- on one hand to investigate the long term evolution of the superficial fissures, revealed in the preceding test, so to assess, through a comparison of the results, the effects produced by the accelerated cycles simulating the climatic excursions;
- on the other to control the appearance of new decay forms resulting from continuous differential deformations (elongations and contractions) particularly of the superficial layer. Part of the observations was also to detect possible decohesions between the shotcrete layers.

Table 1: Phases of thermohygrometric cycle

Cycle Phases	Air condition gaps		Time (h)
	Temperature (°C)	R. Humidity (%)	
HEATING (HOT AIR)	± 05 — 60	± 90 — 30	3
HEAT MAINTENANCE	± 60 — 60	± 30 — 40	1
WETTING (HOT STEAM)	± 60 — 45	± 40 — 80	1
COLD AIR CHANGE	± 45 — 20	± 80 — 70	3
DAMP COOLING	± 20 — 05	± 70 — 95	4

4 Samples production and the climatic chamber

The samples had rectangular shape and measures 30 x 60 cm. For each system 16 samples were produced: they were prepared with the 4 types of shotcrete, each of which was reproduced on 4 samples (4 series). Each panel has a code name consisting of 3 letters, indicating respectively: construction system (R = Rein, M = Monolite), additive with mix-design used (F = optimized with plastic fibres, S = optimized with silica smoke, E = optimized with expansive agent, T = standard), seasoning conditions (V = hot and ventilated seasoning).

To perform the accelerated ageing test, a **climatic chamber** (2.00 x 1.50 x 0.90 m.) was built with an equipment unit. This chamber is able to implement temperature and relative humidity cycles automatically and is structured so as to house the samples vertically closing its lateral sides (Fig. 3). The chamber has 4 openings in the front side, 2 of which with double glazing portholes to visual control.

During the cycle 5 distinct phases (3 hot, 2 cold) was alternating with sudden temperature and relative humidity changes.

The chamber is heated by an air-heating device installed outside the chamber. A cooling air system with suction impeller performs cooling phases. Two air exhausters are placed so that to put cold air into the chamber and expel hot air out through an automatic-opening chimney put on top. A hot vapour generator ensures hot humidification. During the cycles humidification and heating took place simultaneously obtaining a state of damp heat. To obtain low temperature

humidification no device was used, in that cold vapour is spontaneously generated by condensation during the cooling phase.

The climatic chamber is equipped with a switch-board monitor controlling climatic conditions of the air inside it (T, RH).

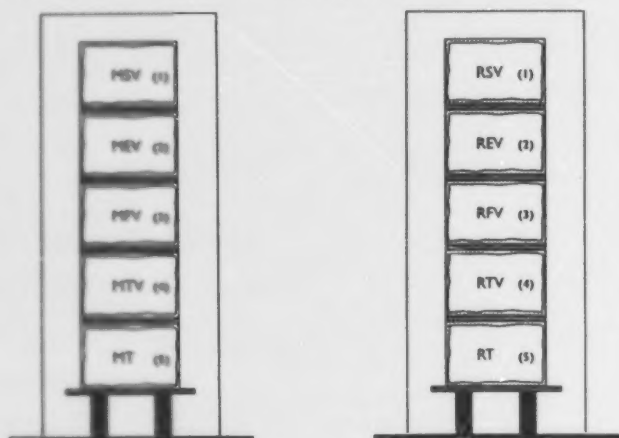


Fig. 3: Panel samples on the lateral walls of the climatic chamber (left - Memolite samples, right - Rein samples)

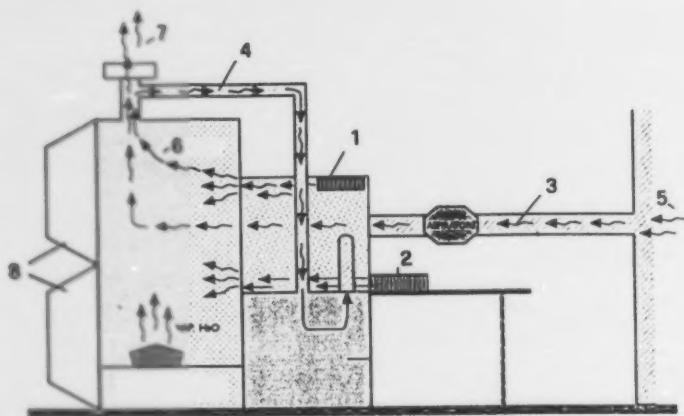


Fig. 4: Working scheme of the climatic chamber - vertical section (1: cooling system, 2: air-heating carrier, 3: air change, 4: cold air recirculation, 5: external air inlet, 6: cold air, 7: hot air outlet, 8: openings)

5 Measurement and investigation techniques

Accelerated tests were carried out along with a visual and instrumental investigation on decay mechanisms at short and long term.

The superficial state was described by the fissures width and length appeared in consequence of the conglomerate setting phase in hot and dry atmosphere. Measures were estimated with a centesimal comparison microscope with sensitivity of 1/100 mm. The data were subsequently compared with those obtained on the same samples at the end of the accelerated ageing test. With the compared analysis it was possible to control the superficial fissure evolution, previously observed, in terms of length and width variations, and to verify the appearance of other defects (Fig. 5).

The adhesion state evolution between layers were studied with the Infrared thermography. Thanks to this diagnostic non-destructive technique it was possible to put forward some hypothesis on the state of the layers, analysed as macroscopic or uniformly distributed (not localized) defects. At the beginning and end of the cycles on the ageing test, the superficial temperatures and their distribution on external surface were measured. Chromatic maps of surface temperatures can be processed by the measurement system (Fig. 6).

The investigation was performed after heating the air inside the chamber to a temperature conformed for the survey (about 80 °C), so to generate, inside the chamber, a thermal flux from the internal surface to the external one (investigated). The thermographies executed in both phases were aimed at observing, through a qualitative images comparison, possible alterations induced by thermal stresses on the internal cohesion state between layers.

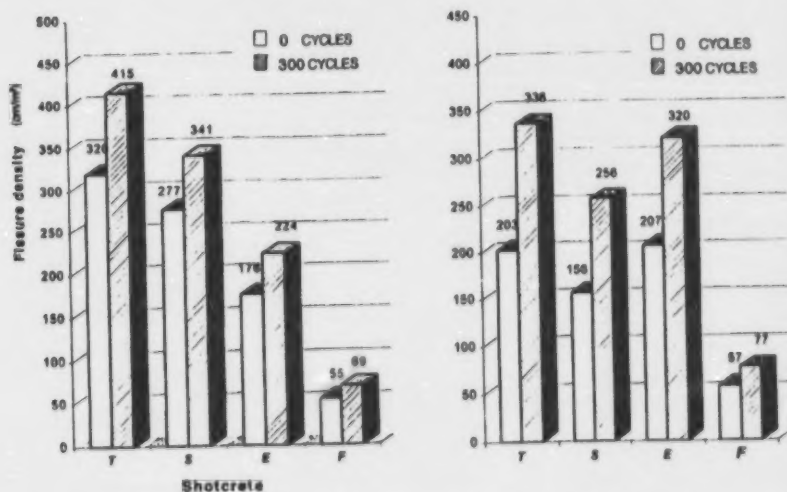


Fig. 5: Evolution of the superficial fissure density at the end of thermal cycles (left - Rein samples, right - Monolite samples)

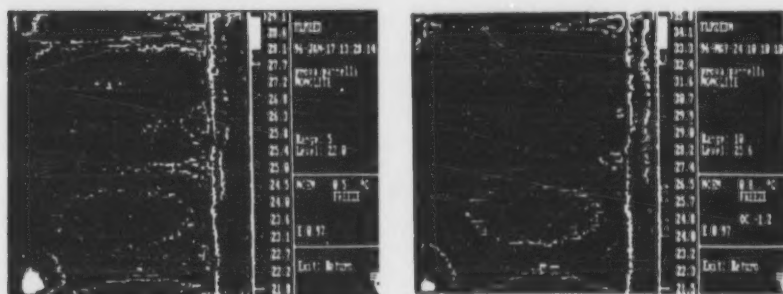


Fig. 6: Thermographic images of some samples at the beginning and the end of the ageing (left - 0 cycles, right - 300 cycles) - AGA Termovision 870 system

Finally, a test was given about the *Laser-Doppler vibrometer* potentiality as an innovative investigation instrument on the adhesion state between layers (Paone 1994). This technique, non-invasive and non-destructive, was recently experimented in the restoration to study superficial peelings of ancient plasters. During the test execution the vibration speeds of the investigated body (in this case the external layer) was measured in consequence of impulse excitation caused by a shaker.

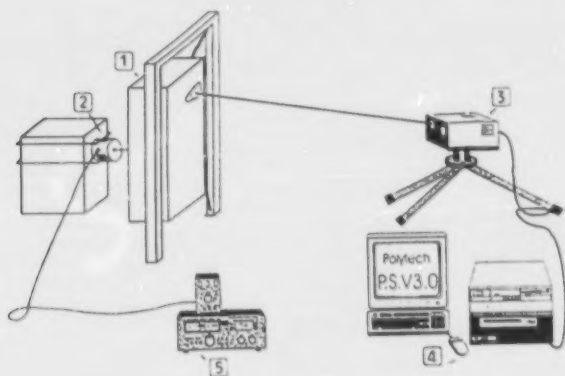


Fig. 7: Equipment used for vibrometric test (1: sample, 2: shaker, 3: Laser-Doppler vibrometer, 4: data acquirer, 5: signal generator)

Generally, the excitation is produced by an acoustic exciter (as a loudspeaker) so that there's no instrumental contact with the surface (non-intrusive). Speed amplitudes processing, relevant to the various points of the surface, furnished a chromatic bidimensional map, where it was possible to observe mobility areas associated with peelings. The test was carried out on a reference sample, on which the peeled parts of the superficial layer were known prior, since they were clearly visible.

Though this technique is still in course of developing, what makes it innovative is its vibration measuring system (laser vibrometer). Besides making the technique absolutely non-intrusive, the laser vibrometer can be used for remote investigations, facilitating the study of elements difficult to access and avoiding the scaffolding use.

6 Results and conclusions

- Because of the thermal actions of the accelerated tests (fissure - ageing), both panel samples showed the forming of an initial fissure state and a time evolution of the fissure density, apart from the panel or shotcrete type. The fissuration tendency, differentiated according to the shotcrete type (T, F, S, E), is qualitatively similar after the tests on both sample series. The fissure evolution caused by the ageing cycles was characterized by a growth of the previously formed fissures and in some cases by the appearance of one else new.
- The increase (%) in fissure density showed a reversal of trend in the Rein and Monolite samples. Under the action of cycles (2° test), the superficial fissure state increased more in the Monolite than in the Rein serie, this last one characterized by a greater initial decay after the fissure test (1° test) instead.
- The use of polypropylene-fibres reinforced shotcrete (F), even in a small quantity (0.9 kgm^{-3}), involved the forming of a lesser fissure density in the initial phase and a lesser tendency to evolution in the ageing phase. Standard shotcrete (T) showed a higher fissure density in both series. The density ratio between these two shotcretes types (T, F) is approximately equal to 5. The shotcrete types (with silica smoke, S - with expansive agent, E) had an intermediate behaviour, showing a sufficiently similar fissure tendency.
- The results achieved highlighted the importance of a correct setting phase control. A bad initial seasoning influenced negatively the decay evolution due to thermal cycles. The data proved that the study of mixes according to the recent rules was valid, since the proposed optimized shotcretes (F, S, E) behaved better than standard traditional shotcrete (T).
- The thermographic investigation on the superficial layers of the samples showed no significant alteration in the internal state in terms of macroscopic local defects (peelings, decohesions) after the ageing test. Unexpectedly, the analysis on the uniformly distributed state, through thermal fluxes comparison in the time, showed an improvement in thermal performances of the samples. The ageing cycles were expected to compromise the performance in terms of thermal insulation; on contrary, a thermal resistance increment was observed, probably due to the shotcrete drying process triggered by the hot cycles.
- The Laser-Doppler vibrometric test, even though it was only an approach to this innovative diagnostic method, proved that this technique is greatly effective and has considerable potentialities. It seems that its application can be extended in building to study the cohesion state of multilayer panels of building in service and, in general, of a stratiform building structure.

- The climatic chamber proved to be suitable for submitting the samples to climatic excursions. At the present the cooling system has been enhanced and can reach temperature of -15°C in $4 + 6$ h at full load. But some corrective measures are necessary to improve insulation in points of thermal leakage (edge effects).

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18 MULTI-HIERARCHICAL DURABILITY ASSESSMENT OF EXISTING REINFORCED-CONCRETE STRUCTURES

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Abstract

Durability of an existing reinforced concrete structure is one of the most important problems for its users and owners. However, no systematic durability assessment approach exists currently. The assessment results vary in relation to the knowledge, expertise, and partiality of the assessor. Additionally, most assessment may not focus on all the key system parameters that affect structural durability. This paper aims at establishing a systematic durability assessment approach for R.C. Structures. The development of such an approach would help to identify all major parameters that affect structural durability and in addition, would enable assessors to collect both qualitative as well as quantitative information in a standard format for any given reinforced concrete structure. Furthermore, such a systematic assessment approach would provide a fair and objective way for analyzing and dealing with data collected by field inspection. Firstly, by Saaty's ratio scaling method, which scientifically quantifies the subjective information of expert judgments. Secondly, on basis of the properties of entropy, which combines the subjective information of expert judgments with the intrinsic information of assessment parameters. Then, taking gray relation grade as criterion, which obtains the deterioration degree and deterioration rate for a given structure. Finally, based on the assumption that deterioration varies with time exponentially, it gives the present performance and residual service life for any existing reinforced concrete structure.

Keywords: reinforced concrete structure, Analytic Hierarchy Process, information entropy, gray relation grade, residual service life prediction

1 Introduction

Durability is one of the main features of an existing reinforced concrete structure, and it is supposed to be assessed regularly in case failure occurs. But unfortunately, no systematic durability assessment approach has been established up to now. In finding such a systematic approach, there are three problems to be solved: how to unify durability assessment with residual service life prediction, how to use the knowledge on durability to identify the key system parameters and what mathematical methods to use to guarantee the objectivity of assessment result. This paper presents a solution to each of the above problem. Firstly, it suggests that two systems, namely deterioration degree system (DDS) and deterioration rate system (DRS), be established for an existing structure. I provide the key parameters, applicable in general cases, for each system. Finally, using Analytic Hierarchy Process (AHP), Information Entropy Formula and Gray Relation Grade Theory, it minimizes the partiality of the assessor to a large extent. Durability of an existing R.C. structure is characterized as two complex systems with each comprising many interrelated parameters. All these parameters are arranged into different levels and grouped into subjective/qualitative and objective/quantitative parameters. The former cannot be known precisely, or in words of Gray Theory, they are gray. Therefore, it is in reason to use AHP and Gray Theory in durability assessment.

2 Steps of multi-hierarchical durability assessment

2.1 Identification of the key parameters

For an existing reinforced concrete structure, there are generally two sets of system parameters: One is for DDS and the other for DRS. Each set consists of a recursive hierarchy composed of three levels: structure, component and factor.

The DDS parameters and DRS parameters are listed in Table 1 (Wang 1996).

Table 1: Key parameters for DDS and DRS

Structure Level	Component Level	Factor Level	
DDS	Beam, Column, Wall, Foundation, and others.	Qualitative/Subjective	Concrete longitudinal cracking, scaling and peeling; AAR and corrosion marks and so on.
		Quantitative/Objective	Percentage of steel corrosion, percentage of strength reduction of concrete, concrete carbonation depth and so on.
DRS	Beam, Column, Wall, Foundation, and others.	Qualitative/Subjective	Climate of service area, including temperature, humidity, concentration of harmful gases, frequency of freeze-thaw alternation, concentration of corrosive materials, corrosion protective measures, and so on.
		Quantitative/Objective	Intensity of polarized electricity, potential of steel corrosion, polarized impedance and so on.

2.2 Determination of gray relation coefficients

2.2.1 Value Assignment of Data Set

It can be seen from Table 1 that both systems have qualitative as well as quantitative parameters. According to the maximum resolving power of the human brain, qualitative parameters are graded into five ranks, which are valued 5, 4, 3, 2, and 1. Usually, a qualitative parameter is evaluated through expert judgements, while a quantitative parameter is determined by measurements. Thus, one can obtain the values of all parameters subordinate to one common higher-level parameter. All these values constitute the practical set, expressed as

$$C = (C_1 \ C_2 \ \dots \ C_n)^T,$$

where: C_j = is the practical value of the j th parameter.

Each parameter C_j has a predetermined optimal value, which is denoted by C_j^* . All the optimal values constitute the optimal set, expressed as $C^* = (C_1^* \ C_2^* \ \dots \ C_n^*)^T$, where C_j^* is the optimal value of the j th parameter.

Through comparison of the practical set to the optimal set, the situation of a given structure can be determined.

2.2.2 Data Normalization

In many cases, the units and magnitudes of different parameters are incompatible, so it is necessary to normalize the data.

Suppose C_j varies in the range $[C_j^L, C_j^U]$, then its normalized value F_j is

$$F_j = \frac{C_j^U - C_j}{C_j^U - C_j^L} \quad (\text{for positive parameters}) \quad (1)$$

$$F_j = \frac{C_j - C_j^L}{C_j^U - C_j^L} \quad (\text{for negative parameters}) \quad (2)$$

Where negative parameter is defined as one with which deterioration increases. Obviously, $F_j \in [0,1]$ and has no dimension.

The normalized value of any parameter is 0. This can be derived from equation (1) and (2). There are two cases. (1) For positive parameter, $C_j^* = C_j^U$ and (2) for negative parameter, $C_j^* = C_j^L$. Both cases lead to $F_j^* = 0$, which indicates that the optimal set has a correspondence with the situation of no deterioration.

A number of engineering examples suggest varying value ranges for the parameters and optimal values are given in Table 2 with asterisks, while the others are the worst. For DDS, the worst values correspond to the condition that structural load carrying capacity decreases by 10%. For DRS, the worst values correspond to the condition that structural deterioration rate reaches the tolerance (preset as 1% per year in this paper). Either situation implies the failure of a structure.

Table 2: Varying ranges and optimal values of some parameters

Varying Range	Qualitative Parameter	Quantitative DD Parameters			Quantitative DD Parameters	
		Crack Width (mm)	Percentage of Steel Corrosion (%)	Percentage of Concrete Strength Reduction (%)	Steel Corrosion Rate (%/year)	Deterioration Rate (%/year)
Upper Bound	5*	0.6	5	10	0.5	1
Lower Bound	0	0*	0*	0*	0*	0*

2.2.3 Calculation of Gray Relation Coefficients

Using the normalized optimal set as a reference of the normalized practical set, the gray relation coefficient between F_j and F_j^* , denoted by β_j (Lu 1998), can be calculated by:

$$\beta_j = \frac{0.5}{1.5 - F_j} \quad (3)$$

2.3 Determination of parameter weight

2.3.1 Initial weight

(1) Formation of Judgement Matrix

By comparing the importance of all parameters subordinate to one common higher parameter, one can obtain the judgement matrix. Generally, if C_j ($j=1,2,\dots,n$) are all subordinate to B , the judgement matrix is an n -D matrix, as listed in Table 3.

In Table 3, a_{ij} indicates the relative importance of parameter i vs. parameter j . To quantify a_{ij} , one can use Saaty's Ratio Scaling Method (See Table 4) (Chen and Lu 1998). (2) Calculation of Maximum Eigenvalue and Eigenvector of Judgement Matrix Computational methods for the maximum eigenvalue and its corresponding eigenvector have been introduced in many algebraic books, so unnecessary details are not given here.

Table 3: General form of judgement matrix

B_i	C_1	C_2	...	C_n
C_1	a_{11}	a_{12}	...	a_{1n}
C_2	a_{21}	a_{22}	...	a_{2n}
...	...	...	...	...
C_n	a_{n1}	a_{n2}	...	a_{nn}

Table 4: Judgement scales and meanings

Scales	Meanings
1	One parameter is as important as the other
2	One parameter is slightly more important than the other
3	One parameter is evidently more important than the other
4	One parameter is intensely more important than the other
5	One parameter is extremely more important than the other
reciprocal	$a_{ji}=1/a_{ij}$

(3) Checkup of Judgement Matrix Consistency

The consistency index for an n -D matrix is determined by

$$CI = \frac{\lambda_{\max} - n}{n - 1} \quad (4)$$

where $\lambda_{\max}$ = maximum eigenvalue, n = number of matrix dimension.

It must be pointed out that CI is function of n . That is to say, matrices with the same $\lambda_{\max}$ but different n values have different allowable consistency. Therefore, a new parameter RI, called average random consistency index, is introduced. RI can be assigned according to Table 5.

Table 5: Average random consistency index RI

n	3	4	5	6	7	8	9
RI	0.58	0.90	1.12	1.24	1.32	1.41	1.45

1-D or 2-D matrix always has complete consistency, so it is unnecessary to take consistency checkup. When n is larger than 2, the consistency can be checked up by the ratio

$$CR = \frac{CI}{RI} \quad (5)$$

If $CR < 0.10$, the judgement matrix is deemed to have satisfying consistency, otherwise it should be modified until it arrives at satisfactory consistency.

On condition that the judgement matrix has satisfactory consistency, its eigenvector corresponding to the maximum eigenvalue is the initial weight W .

2.3.2 Parameter entropy

Initial weight reflects only the subjective expert experience. In most cases, parameters' magnitude also has an influence on the weight. Therefore, it is required that initial weight be modified by parameters' intrinsic information. This modification can be realized by introducing information entropy, which is formulated as (Lu 1998)

$$E_j = - \left(F_j / \sum_{j=1}^n F_j \right) \ln \left(F_j / \sum_{j=1}^n F_j \right), \quad j = 1, 2, \dots, n \quad (6)$$

where E_j = entropy reflecting the importance of F_j .

2.3.3 Final weight

The final weight is given by

$$\omega_j = \frac{E_j W_j}{\sum_{j=1}^n E_j W_j}, \quad j = 1, 2, \dots, n \quad (7)$$

It should be shown that ω_j satisfies $0 \leq \omega_j \leq 1$; and $\sum_{j=1}^n \omega_j = 1$.

2.4 Calculation of assessment results on component level

Any assessment result on component level is actually weighted gray relation grade, which is given by:

$$S = \sum_{j=1}^n \omega_j \beta_j \quad (8)$$

The geometrical meaning of S is the distance between the practical set and the optimal set. The larger the value S , the closer is the distance between practical set and optimal set.

2.5 Calculation of assessment result on structure level

In this step, assessment results on the component level are taken as parameter values on structure level. Structure assessment is carried out in the same procedure as the component assessment. The assessment result includes two assessment values: One is the DDA value, R , and the other, DRA value, $\bar{R}$.

2.6 Calculation of DD and DR on structure level

R and $\bar{R}$ are weighted gray relation grades between the practical set and optimal set for DDS and DRS, respectively. They can be used to countercalculate the equivalent normalized values. From equation (3), two equivalent normalized values F and $\bar{F}$ are derived as follows

$$F = 1.5 - \frac{0.5}{R}, \quad \bar{F} = 1.5 - \frac{0.5}{\bar{R}} \quad (9)$$

The DD and DR on structure level are given by:

$$\bar{O} = F \times 10\%, \quad \Phi = \bar{F} \times 1\% / \text{year} \quad (10)$$

in which Φ = DD on structure level, $\bar{\Phi}$ = DR on structure level.

2.7 Prediction of structural service life

Suppose a structure has serviced for T years, during which it has undergone N repairs. If the i th repair occurred in the T_i th year, and the ratio of the added load carrying capacity to the initial load carrying capacity is η_i , then based on the assumption that deterioration varies with time exponentially, equations (11)(12) are tenable (Lu 1998):

$$\alpha T^\beta + \sum_{i=1}^N \eta_i \alpha (T - T_i)^\beta - \sum_{i=1}^N \eta_i = \Phi \quad (11)$$

$$\alpha \beta T^{\beta-1} + \sum_{i=1}^N \eta_i \alpha \beta (T - T_i)^{\beta-1} = \Phi \quad (12)$$

where α, β are called deterioration coefficient and deterioration exponent, respectively.

The residual service life of a structure, denoted by T_r , can be obtained from the equation

$$\alpha (T + T_r)^\beta + \sum_{i=1}^N \eta_i \alpha (T + T_r - T_i)^\beta - \sum_{i=1}^N \eta_i = 10\% \quad (13)$$

It can be known from equation (13) that residual service life is function of the service time, maintenance record, and assessment results.

3 Illustrative example

A one-bay and two-story portal frame to be assessed is shown in Figure 1. The frame is located in a coastal area and has been used for 10 years. Before being assessed, the frame underwent no repair. The data collected by field inspection are shown in Table 6.

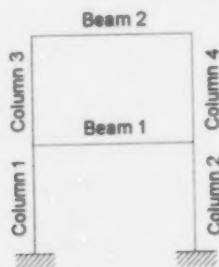


Fig. 1: One-bay and two-story frame

Table 6: Data collected by field inspection

Component			Beam 1	Beam 2	Column 1	Column 2	Column 3	Column 4
DDS	Qualitative	Overall Appearance	4.4	4.1	3.8	3.6	3.9	4.1
	Quantitative	Crack Width (mm)	0.09	0.12	0.21	0.24	0.27	0.21
		Percentage of Steel Corrosion (%)	1.1	1.2	1.6	1.8	1.2	1.0
		Percentage of Concrete Deterioration (%)	1.5	2	2.5	2.5	2.0	2.0
DRS	Qualitative	Environmental Condition	3.6	3.4	3.0	3.0	3.6	3.7
	Quantitative	Steel Corrosion Rate (%/year)	0.05	0.07	0.12	0.14	0.09	0.06
		Deterioration Rate of Concrete Strength (%/year)	0.30	0.40	0.50	0.50	0.40	0.36

3.1 Determination of initial weight

The results are shown in Figures 2 and 3.

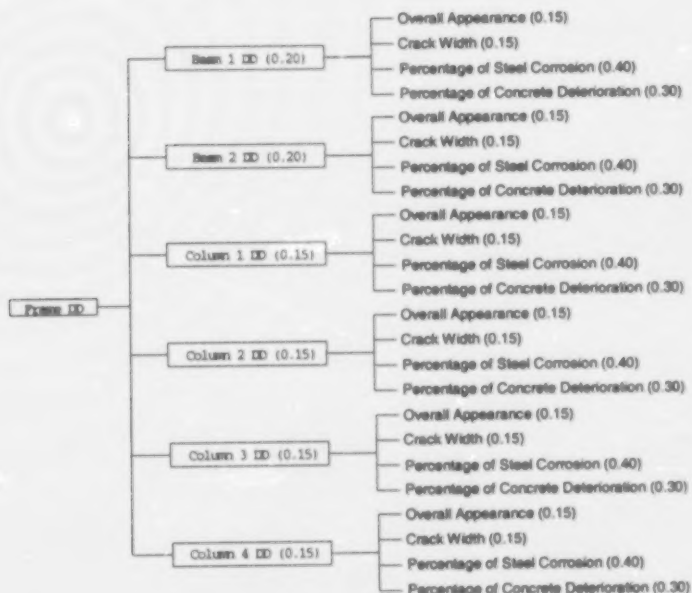


Fig. 2: Deterioration degree system and initial weight

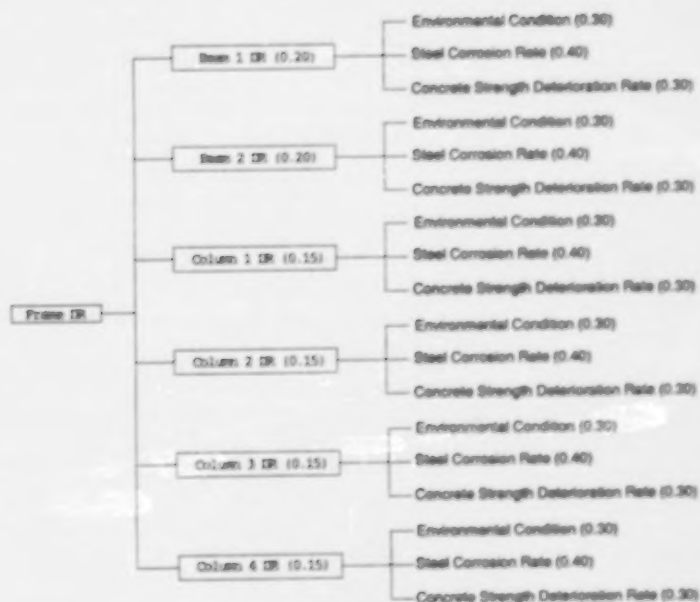


Fig. 3 Deterioration rate system and initial weight

3.2 Normalization of parameters

The results are listed in Table 7.

Table 7: Normalized values of parameters

Component			Beam 1	Beam 2	Column 1	Column 2	Column 3	Column 4
DDS	Qualitative	Overall Appearance	0.12	0.18	0.24	0.28	0.22	0.18
	Quantitative	Crack Width (mm)	0.15	0.20	0.35	0.40	0.45	0.35
		Percentage of Steel Corrosion (%)	0.22	0.24	0.32	0.36	0.24	0.20
		Percentage of Concrete Deterioration (%)	0.15	0.20	0.25	0.25	0.20	0.20
DRS	Qualitative	Environmental Condition	0.28	0.32	0.40	0.40	0.28	0.26
	Quantitative	Steel Corrosion Rate (%/year)	0.10	0.14	0.24	0.28	0.18	0.12
		Deterioration Rate of Concrete Strength (%/year)	0.30	0.40	0.50	0.50	0.40	0.36

3.3 Assessment Results

The assessment results are shown in Table 8.

Table 8: Assessment results of components and frame

Component		Beam 1	Beam 2	Column 1	Column 2	Column 3	Column 4
DDS	Final Weight	0.1952	0.1978	0.1526	0.1544	0.1512	0.1489
	Component Assessment Value	0.3778	0.3888	0.4148	0.4262	0.4057	0.3921
	Frame Assessment Value	0.3995					
	Equivalent Normalized Value	0.2484					
DRS	Final Weight	0.1949	0.1992	0.1545	0.1551	0.1491	0.1471
	Component Assessment Value	0.3941	0.4138	0.4470	0.4513	0.4119	0.3999
	Frame Assessment Value	0.4186					
	Equivalent Normalized Value	0.3055					

3.4 Analysis results

- (1) From Table 8, it is seen that deterioration degrees of components are in the following order:

Beam 1 < Beam 2

Column 4 < Column 3 < Column 1 < Column 2

- (2) From Table 8, it is seen that deterioration rates of components are in the following order:

Beam 1 < Beam 2

Column 4 < Column 3 < Column 1 < Column 2

- (3) Equivalent normalized DD value of frame is 0.2484. This indicates present DD of the frame has reached 24.84% of the tolerable DD. Load carrying capacity of the frame has decreased by $24.84\% \times 10\% = 2.48\%$.

- (4) Equivalent normalized DR value of frame is 0.3055. This indicates present DR of the frame is 30.55% of the tolerable DR, i.e. present decreasing rate of load carrying capacity is $30.55\% \times 1\% = 0.306\%/year$.

- (5) From equations (11)(12), we have $\alpha = 0.0146$, $\beta = 1.234$. Then from equation (13), the residual service life of the frame is determined as 21 years.

4 Conclusions

By now we have established a systematic durability assessment approach for reinforced concrete structures. This approach provides a fair and objective way for analyzing and dealing with data collected by field inspection. The advantages of the approach lie in treating durability of a structure as two multi-hierarchical systems and combining experts' grading with parameters' entropy. By carrying out DDA and DRA, we can obtain more information, such as present structural performance and residual service life. Though the proposed approach takes quite a few steps, yet it is easy to be programmed.

5 References

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19 CHARACTERISATION OF IMPACT DAMAGE IN FIBRE-REINFORCED CEMENT CORRUGATES

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Abstract

In this paper, FRC corrugates were simple-supported and subjected to varying levels of impact energy by dropping a sand bag from different heights. The resulting impact damage was found to be associated with distinct crack progression stages, varying in length and shape depending on the level of impact energy imposed. Slower progression of cracking and higher impact energies at failure were found to be associated with increased corrugate spans.

By correlating the crack progression stages corresponding to corrugates tested at 575 mm and 700 mm impact spans to dynamic mid-span deflection measurement, three impact damage zones were proposed in which impact damage could be characterised by specifying the crack progression stage(s) corresponding to a dynamic deflection range. Using this concept, the allowable "safe" impact energy corresponding to a particular corrugate span was estimated as 60% of the failure impact energy.

Keywords: fibre-reinforced material, corrugates, impact behaviour.

1 Introduction

Fibre-reinforced cementitious (FRC) corrugate is an autoclave-cured medium density highly impact resistant laminate composed mainly of cement, ground silica and cellulose fibres. It is being used extensively as a roof and facade element exhibiting dynamic resistance to impact loading such as hail and nail shooting fixation (Sayer et al. 1992).

Due to its good strength/weight and stiffness/weight ratios, FRC corrugated sheeting can also be used as a permanent formwork element, conforming to the structural requirements of AS 3610 - (1995) "Formwork for Concrete", in

concrete-topped suspended flooring. In this application, corrugates spanning between closely spaced steel or concrete beams form a temporary decking upon which shrinkage reinforcement and concrete topping (up to 100 mm thick) are laid.

2 Research objective

Although there is no requirement in AS 3610 for impact assessment of permanent formwork elements during floor construction, there is a need to assess the extent of impact damage in FRC corrugates used as permanent formwork. Such damage could be initiated in the early stages by falling objects and traffic of tradesmen during installation of the decking, or at later stages due to dynamic loading caused by pumping and spreading of the concrete topping.

The aim of this study is to characterise the impact damage in FRC corrugates subjected to soft body impact. A methodology needs to be developed for quantifying the extent of impact damage, which is normally assessed visually. Also, an upper limit needs to be established for the allowable "safe" impact energy to provide an acceptable factor of safety against failure in corrugates used as permanent formwork.

3 Experimental program

3.1 Impact testing set-up

A single corrugate (6.5 mm thick, 1085 mm long, 50 mm pitch, 150 mm peak spacing) was simple-supported centrally between two inverted, 3650 mm long, pre-stressed concrete "T" beams, and tested under two impact spans, namely: 575 mm and 700 mm, with corrugation laid perpendicular to the beam direction (Figures 1 and 2).

Impact testing was carried out following the method set out in ASTM E695 "Measuring Relative resistance of Wall, Floor and Roof Construction to Impact Loading" for a Horizontal Specimen (Floor or Roof) - (1985). Varying levels of "soft body impact" energy were achieved by dropping a 272 N (60 lb) impact bag from different heights. The bag was suspended over the centre of the corrugate from a quick release mechanism fitted to a crane hook, and measuring blocks were used for setting of the required height (Figure 3). LVDT's positioned centrally under the corrugate were used to measure the dynamic deflection corresponding to each impact height.

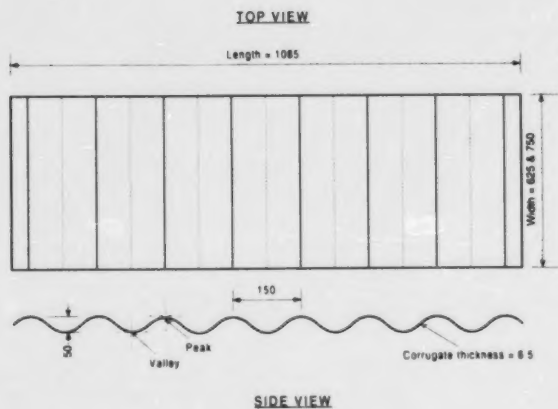


Fig. 1: Corrugate details (dimensions in mm)

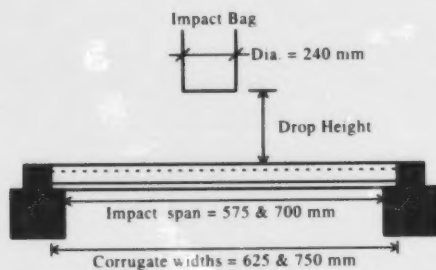


Fig. 2: Impact testing set-up (End View).

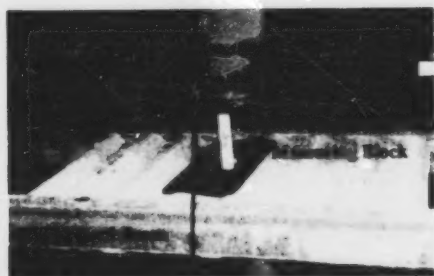


Fig. 3: Impact testing set-up (Side View)

3.2 Impact testing procedures

The impact bag was dropped on air-dry (9% moisture content) FRC corrugates suspended at 575 mm and 700 mm impact spans following the procedures outlined below:

3.2.1 Single - drop impact

The corrugates were subjected to a series of single impact bag drops from various heights, starting from 200 mm and increasing in 50 mm increments up to 500 mm. The extent of damage caused by the impact energy at each drop height was monitored by noting the extent of crack progression and the corresponding dynamic deflection values. A minimum of three corrugate samples were tested for each impact height.

3.2.2 Failure "fall-through" impact

A series of single impact bag drops from various heights were conducted, starting from 500 mm and increasing in 50 mm increments. The height corresponding to "fall-through" impact (Figure 4) was taken as the impact height at failure. A minimum of three corrugate samples were tested at "fall-through" impact height.

3.2.3 Repeated - drop impact

The corrugates were subjected to a series of repeated impact bag drops from fixed heights, starting from 200 mm and increasing in 50 mm increments up to failure. A minimum of three corrugate samples were tested for each impact height.

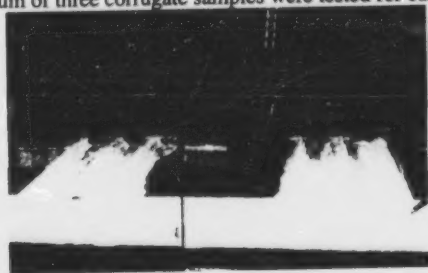


Fig. 4: A "fall-through" failure under impact

4 Results & discussion

4.1 Crack progression stages in corrugates under impact

The progression of cracking in corrugates subjected to increasing impact energies under single-drop impacts is shown in figure 5. Five distinct crack progression stages could be differentiated. The impact-affected area was taken as the area bound by the two neighbouring peaks sharing the impact (figure 5).

Stage 1: Localised cracking

A single horizontal crack is formed in the valley between the two corrugate peaks directly hit by the impact bag.

Stage 2: Widening/branching of localised crack

A widening of the crack is observed coupled with vertical branching. Cracking is still confined to the area directly hit by the impact bag.

Stage 3: Extended cracking

The crack extends to the neighbouring valleys on each side of the impact affected area.

Stage 4: Formation of cross-cracking

The crack branches vertically, mainly in the impact affected area, forming a "cross" shape.

Stage 5: Extension of cross-cracking (Failure)

The cross-shaped crack extends the full width of the FRC corrugate, causing it to fail, i.e. break into pieces.

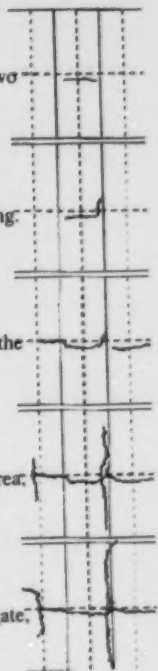


Fig. 5: Crack progression stages in FRC corrugates

In table 1, the crack progression stages corresponding to corrugates suspended at 575 mm and 700 mm impact spans, and subjected to increasing impact energies up to failure, are outlined. The impact energies were calculated by multiplying the bag mass (272 N) by the drop height (meter).

It can be seen that, at low impact energy levels, i.e. 55 Joule to 95 Joule, cracking in the impact affected area appears to be localised in the valley (stages 1 and 2). This is probably caused by the tensile stresses generated upon impact which are confined by the compressive stresses in the neighbouring peaks. With increased impact energy, i.e. 109 Joule to 123 Joule range, progressive reduction in the local stiffness of the corrugate takes place (Sayer et al. 1992), causing cracking to be extended beyond the impact affected area (stage 3), branching into a "cross" formation (stage 4), followed by failure (stage 5).

Table 1: Cracking stages corresponding to corrugates (air-dry condition) tested in single-drop impact at various heights

Impact Height (mm)	Energy (Joule)	Cracking Stage	Description (refer to Fig. 5)
575 mm Impact Span			
200	55	Stage 1	Localised cracking
250	68	Stage 1	Localised cracking
300	82	Stage 1	Localised cracking
350	95	Stage 2	Widening/branching of localised crack
400	109	Stage 3	Extended cracking
450	123	Stage 4	Formation of cross-cracking
500	136	Stage 4	Formation of cross-cracking
700 (*)	190	Stage 5	Extension of cross-cracking / failure
700 mm Impact Span			
200	55	Stage 1	Localised cracking
250	68	Stage 1	Localised cracking
300	82	Stage 1	Localised cracking
350	95	Stage 1	Localised cracking
400	109	Stage 3	Extended cracking
450	123	Stage 3	Extended cracking
500	136	Stage 4	Formation of cross-cracking
800 (*)	218	Stage 5	Extension of cross-cracking / failure

(*) "Fall-through" failure caused by single-drop impact

From the above, it can be suggested that FRC corrugates subjected to drop sand bag (soft body) impact exhibit a progressive and non-catastrophic failure mode which seems to be caused by the effect of corrugation on limiting crack progression.

4.2 Effect of corrugate span on resistance to impact damage

The impact damage trends corresponding to corrugates tested at two impact heights, ie 575 mm and 700 mm, are shown in table 1. It can be noted that, slower progression of cracking at low - intermediate impact energy levels, ie 55 Joule to 123 Joule, and 15% improvement in impact resistance at failure, ie failure energy increase from 190 Joule to 218 Joule, were associated with corrugates tested at the higher impact span, ie 700 mm.

The above trends can probably be explained by relating the impact behaviour of FRC corrugates to the model outlined by (Sayer et al. 1992). In this model, the impact force acting on the corrugate (F) is said to be split between one part (F_1) representing the force generating pure flexural, and another part (F_2) which represent additional membrane forces due to the curvature of the corrugated structure. It follows that, in terms of impact damage, the forces acting upon impact can be interpreted as the force (F_1) which retain the damaging stresses within the corrugate (dashpot action), and the force (F_2) which is released as membrane modes of motion (spring action). It is hypothesised that in the case of corrugates tested at 700 mm in lieu of 575 mm, a larger proportion of the impact force will be compensated as membrane forces (F_2) due to the enhanced "spring effect", leaving less of the (F_1) force component, and consequently causing less impact damage.

4.3 Characterisation of impact damage using proposed impact damage zones

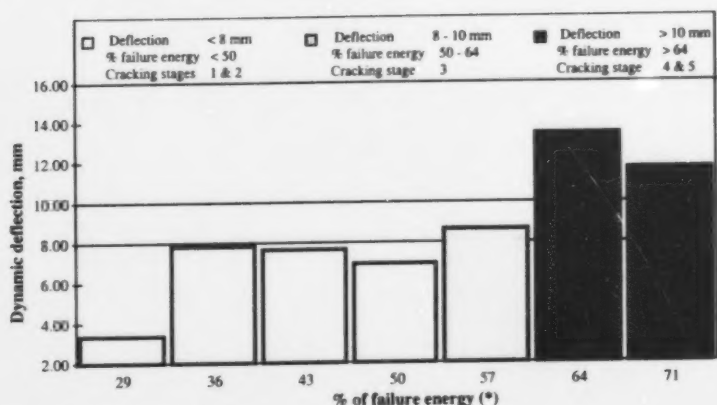
The mid-span dynamic deflection values corresponding to corrugates tested at 575 mm and 700 mm spans at various impact energy levels (expressed as % of failure energy) are presented in table 2 and shown in figures 6 and 7. It can be seen that, three dynamic deflection ranges, namely: (< 8 mm), (8 mm to 10 mm), and (> 10 mm), could be identified which closely followed the corresponding crack progression stages shown in table 1.

Based on the above, it is proposed that three impact damage zones can be identified in FRC corrugates subjected to soft body impact. For each impact zone, a dynamic deflection range is nominated which corresponds to a level of impact damage described in the form of crack progression stage(s).

The impact damage zones with corresponding dynamic deflection ranges and crack progression stages are presented in table 3. It is suggested that characterisation of impact damage in corrugates can be established by relating the deflection measurement upon impact to an impact damage zone, ie dynamic deflection range, and consequently, a crack progression stage.

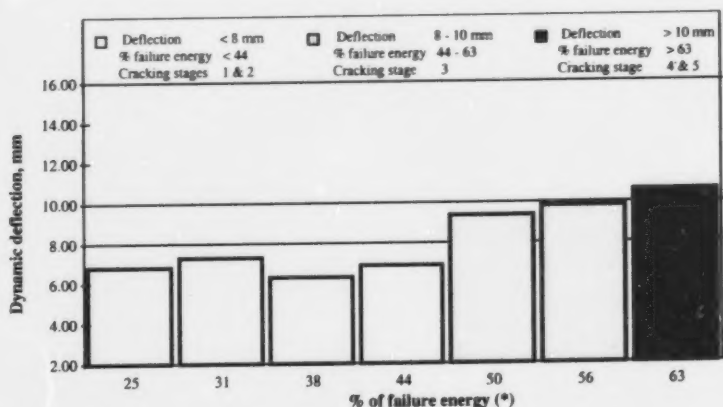
Table 2: Dynamic deflection values (avg. of 3) corresponding to corrugates (air-dry condition) tested in single-drop impact at various heights

Impact Height (mm)	Impact Level (% failure energy)	Avg. Dynamic Deflection (mm)	Coefficient of Variation
575 mm Impact Span			
200	29	3.32	18
250	36	7.84	3
300	43	7.65	23
350	50	6.94	9
400	57	8.66	26
450	64	13.40	19
500	71	11.65	14
700 mm Impact Span			
200	25	6.83	27
250	31	7.31	41
300	38	6.32	18
350	44	6.86	11
400	50	9.36	7
450	56	9.85	15
500	63	10.60	3



(*) Failure energy = 190 Joule

Fig. 6: Dynamic deflection vs impact energy (normalised as % of failure energy) for corrugates tested at 575 mm impact span



(*) Failure energy = 218 Joule

Fig. 7: Dynamic deflection vs impact energy (normalised as % of failure energy) for corrugates tested at 700 mm impact span

Table 3: Impact Damage Zones as characterised by dynamic deflection values

Impact Damage Zone	Dynamic Deflection Range (mm)	Crack Progression Stage
Zone 1	< 8.0	Stages 1 & 2 (Localised Cracking)
Zone 2	8.0 - 10.0	Stage 3 (Extended Cracking)
Zone 3	> 10.0	Stages 4 & 5 (Cross-shaped Cracking)

4.4 Estimation of allowable impact energy (expressed in % of failure energy)

Using the concept of impact damage zones, it is proposed that zone 2 in table 3, which corresponds to a dynamic deflection range of 8 mm to 10 mm, is nominated as the "safe" upper limit in corrugates undergoing soft body impact. From figures 5 and 6, it can be seen that the upper limit within this range corresponds to the transition from stage 3 damage (extended cracking) to stages 4 & 5 (cross shaped cracking and failure), which ranges between 57% to 63% of failure impact energy.

It is, therefore, proposed that this transition impact energy range, ie average of 60% of failure energy, is considered as "allowable" impact energy, providing a factor of safety of 1.67. This figure corresponds to a maximum dynamic deflection value of 10 mm (Figures 5 and 6) and can be used by engineers to determine the maximum allowable impact energy that can safely be imposed on FRC corrugates within the impact span range undergone in this study.

4.5 Impact behaviour under repeated-drop impact

Table 4 outlines the number of repeated impact drops necessary to cause failure from a range of impact height in corrugates tested at 575 mm and 700 mm impact spans. It can be seen that the number of impact drops needed to fail the corrugates was influenced by drop height, with lower height requiring a higher number of drops to achieve failure.

It can also be noted that, similar to the trend observed in 4.2, corrugates with 700 mm impact span sustained a larger number of drops, i.e. exhibited improved impact resistance, compared with those tested at 575 mm span. This improvement in impact resistance was only evident in the 200 mm to 350 mm drop height range (Table 4).

Table 4: Repeated impact bag drops to failure for corrugates tested at 575 and 700 mm impact spans (starting from 200 mm impact height)

Impact Height (mm)	No. Of Sample 1	sample 2	Drops Sample 3	Average Drops
575 mm Impact Span				
200	4	6	5	5
250	3	3	3	3
300	2	3	3	3
350	2	2	2	2
400	2	2	3	2
450	2	2	2	2
500	2	2	2	2
700	1	1	1	1
700 mm Impact Span				
200	5	6	6	6
250	5	5	6	5
300	4	4	4	4
350	3	3	3	3
400	2	2	3	2
450	2	2	2	2
500	2	2	2	2
800	1	1	1	1

The average drops vs impact height trends corresponding to both impact spans are plotted in Figure 8. It can be seen that, for both corrugate spans, the trends appear to be best approximated by a power best-fit line following general empirical relationship shown below:

$$N = A H^B$$

where (N) is the number of drops, (H) is impact drop height in mm, and (A), (B) are factors depending on impact span. The above equation can be used for estimating failure impact energy in corrugates subjected to repeated-drop impact.

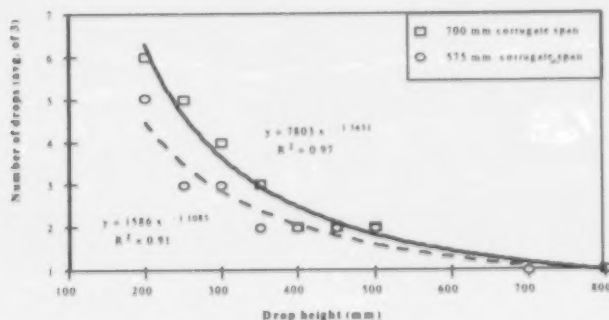


Fig. 8: Average drops vs impact height trends for corrugates tested in 575 mm and 700 mm impact spans under repeated-drop impact

5 Conclusions

Within the range of impact spans investigated in this study, the following findings can be deduced based on the results obtained:

1. Corrugates subjected to soft body impact exhibit a non-catastrophic crack progression trend which can be characterised into five distinct crack progression stages, depending on the impact energy level generated.
2. Slower progression of cracking and improved impact resistance at failure appears to be associated with increased corrugate span under impact.
3. The extent of damage in corrugates could be characterised using dynamic deflection measurement on impact. For this purpose, three impact damage zones were proposed covering the stages of impact damage up to failure. Each zone was correlated to a dynamic deflection range and specific crack progression stage(s).
4. Based on the proposed impact damage zones, a "safe" allowable impact energy in corrugates under impact was estimated as 60% of failure energy, providing a factor of safety against failure of 1.67.
5. A general empirical relationship relating the number of bag drops and the impact height in corrugates under repeated-drop impact was presented. The trend was best approximated by a power best-fit line.

6 References

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20 EFFECT OF WATER STORAGE TIME ON FROST RESISTANCE OF CONCRETE

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Abstract

In high performance concrete (HPC) with low water-cement ratio, cement reaction will cause substantial drying expressed in terms of relative humidity (RH) of the pore water. This phenomenon is called self-desiccation. The RH-values as low as 85% have been measured in dense concretes that have been stored for more than 18 months in water. This means that there is almost no freezable water in the concrete until very low temperatures are reached. It also means that freeze-thaw tests made with young concrete might give results on the unsafe side. In this report some results from a bigger study of the effect of long-term water storage on the frost resistance of HPC are presented. The results show that previously self-desiccated concrete absorbs water when it is stored for a long time in water. Dilation tests during freezing indicate that concrete that was frost resistant when tested during the first months might become non-frost resistant after a long period of water storage. This must be considered in the design of freeze-thaw test methods.

Keywords: Frost resistance, high-performance concrete, self-desiccation, water absorption

1 Introduction

Test standards for frost resistance of concrete normally specifies that the concrete shall be about 28 days old at the start of the test. Besides, it is often specified that the concrete shall be water-stored from the casting until start of the test. It is questionable whether the result of such a freeze-thaw test is always representative for the real structure. In reality many concrete structures are exposed to more than 50 years of continuous water storage in combination with repeated freezing and thawing cycles. Examples are bridge pillars in the splash zone, and hydraulic structures. After some time, the water content in such

structures will probably be considerably higher than in the test specimen. In high performance concrete (HPC) there is a self-desiccation that further lowers the water content in the specimen and thereby increases the frost resistance. One cannot exclude, however, that this self-desiccation gradually disappears in the real structure due to the inflow of water that is not compensated for by desiccation caused by the gradually slower cement reaction.

In this investigation the effect on the frost resistance of long-term water storage before start of the freeze-thaw test was investigated. Water storage was extended to 3 years. The HPC, with and without entrained air, was investigated. The w/c-ratio varied between 0.27 and 0.40. Only a few results are presented here. The rest of the study will be presented in reports from the division.

2 Self-desiccation and its potential effects

The specific volume of chemically bound water in cement paste is only about 0.75 litre/kg (Powers 1962). Thus, when a moisture-insulated cement reacts with water, an air-filled pore space is created. It can be calculated by

$$a = 0.25 \cdot w_n \quad (1)$$

where a (litre) is the air-space created, and w_n (litre) is the amount of chemically bound water. For Portland cement, the maximum amount of chemically bound water when all of the cement has hydrated is about $0.25 \cdot C$ where C (kg/m^3) is the cement content. Then, using the assumption that the degree of hydration α is proportional to the amount of chemically bound water, the air space can be calculated by

$$a = 0.25 \cdot 0.25 \cdot \alpha \cdot C = 0.0625 \cdot \alpha \cdot C \quad (2)$$

where C (kg/m^3) is the cement content in one cubic metre of concrete. Examples: (1) For a normal concrete with the cement content 300 kg/m^3 and the degree of hydration 80% the air space formed is 15 litres/m^3 . (2) For HPC with the cement content 500 kg/m^3 and the degree of hydration 40% the air space is 12.5 litres/m^3 . This means that the self-desiccation expressed in terms of the volume of air space formed is not bigger in HPC than in normal concrete due to the lower degree of hydration reached in HPC. However, in terms of the reduction in RH, self-desiccation is considerably larger in HPC; see below.

This self-desiccation is essential for protection of the concrete against early freezing. In fact, it is the reason why concrete can be placed with success during cold weather conditions. This is further treated by Fagerlund (1997).

For normal weight concrete ($w/c > 0.40$) the self-desiccation will not significantly change the internal relative humidity (RH) of the pore water. The desorption isotherm of such concrete is too steep at high values of RH. This in turn depends on the fairly coarse pore structure of normal concrete. For concrete with lower w/c ratios (HPC), the pore structure is finer, and therefore the sorption isotherm is more horizontal at high RH-values. This means that the internal drying caused by the cement hydration will lead to a significant reduction of the internal RH. For concrete with silica fume this characteristics is even more pronounced due to the more refined pore-structure caused by silica fume. The difference

between normal concrete and HPC is shown in Fig 1.

The reduction in RH will reduce the amount of freezable water. A relation between RH of capillary water and its freezing point is

$$\ln(RH) = \left(\frac{\Delta H}{R \cdot 293} \right) \ln \left(\frac{T}{T_0} \right) \quad (3)$$

where ΔH (J/kmole) is the molar heat of fusion of water ($\Delta H = M(334 + 2 \cdot \theta)10^3$ where θ is the temperature in degrees Celsius), R is the gas constant, T (°K) is the freezing temperature and T_0 (°K) the freezing temperature of bulk water (273 °K). This means that water in equilibrium with 92% RH, 85% RH or 79% RH will not freeze until -10°C, -20°C and -30°C respectively.

Consequently, if an HPC reaches an internal RH of 85% due to self-desiccation there will be no water that can freeze until the temperature is below -20°C.

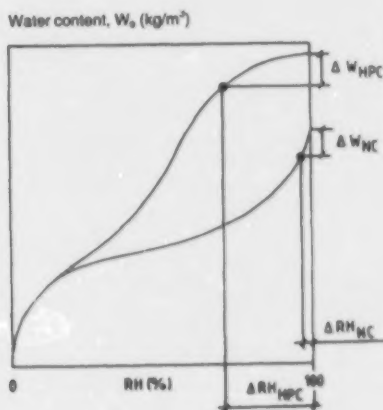


Fig. 1: Shape of the sorption isotherm of normal concrete (NC) and high-performance concrete (HPC) ΔRH is the self-desiccation caused by the hydration ΔW . (Fagerlund 1998)

Theoretically, the self-desiccation will prevail only if the concrete is moisture insulated so that no water can enter the concrete from the surroundings. If the concrete is open to external water, which is normally the case, it will absorb some water, which will raise its internal RH. Tests have shown, however, that a dense concrete can stay dry for a very long time even during very moist conditions. There is even a possibility that a concrete which was once self-desiccated will never become saturated again.

A freeze-thaw test often starts within one or two months after casting of the specimen. This means that one can assume that the internal parts of the freeze-thaw specimens are more or less dry when the test starts. This also applies in cases where the concrete has been stored in water. During the test, a certain moisture ingress might occur, especially if the test is very moist. One cannot exclude,

however, that there is less freezable water in the specimen during the test than what might appear in the real structure. Frost damage will only occur when a critical moisture content is transgressed. Freeze-thaw tests, therefore, always must be made with concrete that have a moisture content that is representative for the prevalent field conditions. A representative moisture condition in HPC can only be reached by prolonged water storage, or by forcing water in by over-pressure. In the actual study, the prolonged water storage was used.

3 Investigations

3.1 Introduction: Tested concrete

Between 1992 and 1998, there has been a large national project in Sweden on high-performance concrete. As a part of this project, 26 different types of HPC specimens were cast in 1995 (Svensson 1995). The water/binder-ratio varied between 0.40 and 0.27 and the amount of air was between 1 and 9%. The cement was a low-alkali ($K_2O+Na_2O=0.6\%$) sulphate resistant ($C_3A=2\%$) standard portland cement. The aggregate consisted of gravel 0-8 mm, quartzite 8-12 mm and quartzite 12-16 mm. The air-entrainment agent used was vinsol-resin. The concretes were cast in plastic pipes with a diameter of 103 mm and a length of 250 mm. Seven specimen of each type of mixture were cured in seven different ways. The cylinders were stored either in limewater or 3% sodium chloride solution. An extensive testing of these cylinders is going on. The testing includes measuring of length change during freeze/thaw cycles due to ice-formation and melting, self-desiccation and field studies of specimens exposed to de-icing salts along a Swedish road. Additional results will be published during 1999. This report contains results from measurements on specimen listed in Table 1. The investigation includes water absorption and freeze/thaw tests of specimen stored in limewater for three years.

Specimen in italics was part of a study of the internal self-desiccation performed by Fagerlund and Yang (Fagerlund and Yang 1998). The last number in the designation is connected to how the specimen was cured. 1 stands for storage in limewater since its de-moulding. 4 signifies that de-moulding was followed by two weeks of membrane curing which then was followed by storage in limewater.

3.2 Self-desiccation

In a study (Fagerlund and Yang 1998) the self-desiccation of high-performance concrete stored for two years in limewater was investigated. The specimens came from the large study described above. At a distance of 70 mm from the cylinder end a slice of 10 mm thickness was sawn. The slice was cut into three layers (inner, middle and outer) and the pieces from each layer were crushed by a hammer and placed in glasstubes. The tubes were put in a climate chamber ($18.2 \pm 0.2^\circ\text{C}$, $62.2 \pm 0.2\%$ RH) for four days before the RH-measurements started by using sensors placed in the tube. The RH was measured after the sensor had come to equilibrium with the air in the tube after about 24 hours. The results are seen in Figures 2 and 3.

Table 1: Tested concretes

Specimen Number	w/c-ratio	Air-content in fresh concrete (%)
0.40-0.0-1	0.40	1.0
0.40-0.5-1	0.40	4.6
0.35-0.0-1	0.35	0.6
0.35-0.4-1	0.35	3.6
0.33-0.0-1	0.33	1.7
0.33-0.3-1	0.33	2.5
0.33-0.4-1	0.33	3.5
0.33-0.9-1	0.33	9.2
0.30-0.0-1	0.30	1.1
0.30-0.3-1	0.30	2.6
0.27-0.0-1	0.27	0.3
0.27-0.3-1	0.27	3.5
0.40-0.0-4	0.40	1.0
0.40-0.5-4	0.40	4.6
0.35-0.0-4	0.35	0.6
0.35-0.4-4	0.35	3.6
0.33-0.0-4	0.33	1.7
0.33-0.4-4	0.33	3.5
0.30-0.0-4	0.30	1.1
0.30-0.3-4	0.30	2.6
0.27-0.0-4	0.27	0.3
0.27-0.3-4	0.27	3.5

The lower the w/c-ratio, the lower is the RH-value inside the concrete. In specimens with silica fume (not shown here), the RH-value was even lower. This is of great importance for the expansions during the freeze/thaw tests of the specimen, which will be presented in Section 3.5.

If this type of concrete is stored in water for a long time, there is a larger possibility that the self-desiccated pores will become filled with water. The concrete has a very low permeability but theoretically water transport does not stop until the moisture content is equal over the entire cross-section. When water reaches the inner parts of the concrete, the self-desiccated pores will be filled. This will lower the possibility of the concrete to withstand freeze/thaw cycles.

3.3 Long-term water absorption in specimens without air-entrainment

The specimens were weighed at ages of 1, 3, 33 and 36 months. The specimens were kept in saturated limewater all the time except during the freeze/thaw tests (see section 3.5) which were conducted in a moisture sealed condition.

The increase in the degree of saturation (ΔS) is calculated according to equation (4) (Fagerlund 1977).

$$\Delta S = \frac{V_w}{V_p} \quad (4)$$

where: V_w (m^3) is the increased volume of water in the specimen related to the volume of water after four weeks of water storage, and V_p (m^3) is the calculated paste pore volume; the calculation is based on equation (5).

$$V_p = C \left(\frac{w}{C} - 0.19 \cdot \alpha \right) \quad (5)$$

where: C (kg/m^3) is the amount of cement, w (kg/m^3) is the amount of mixing water and, α is the degree of hydration which is supposed to be 0.8 for all of the specimens.

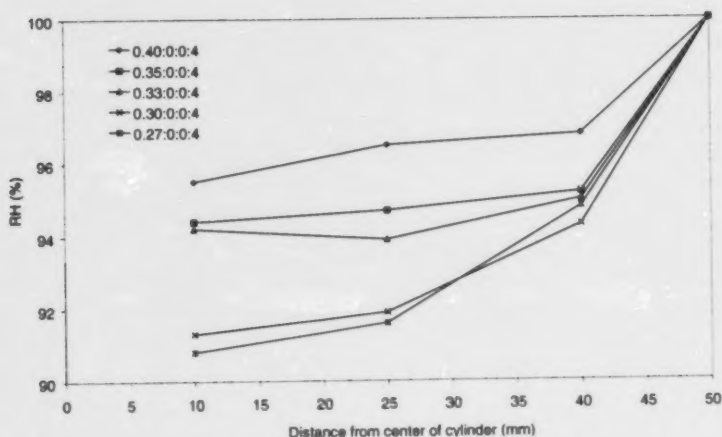


Fig. 2: The distribution of RH in specimen without air-entrainment

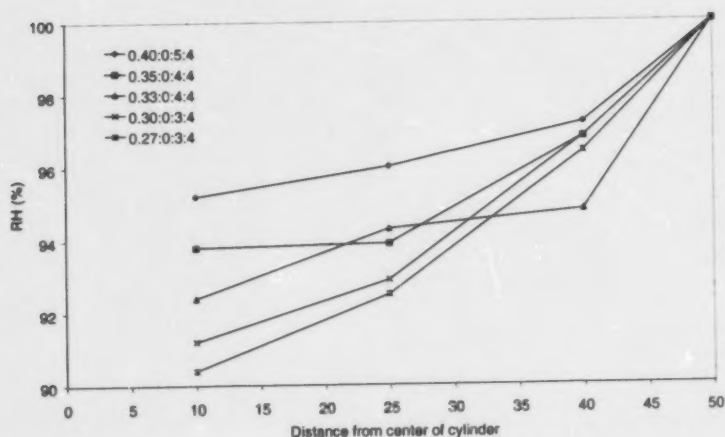


Fig. 3: The distribution of RH in specimen with air-entrainment

All of the specimens show an increase in the degree of saturation (Fig 4). The lowest w/c ratios show the largest absorption, which might depend on the fact that they had the largest self-desiccation. For these concretes, the rate of water absorption is still high. This indicates that some of the pores created by the self-desiccation have started to absorb water. When the degree of saturation exceeds the critical degree of saturation, the concrete will be damaged during a freeze/thaw attack. If the water absorption continues at this rate, that degree of saturation will be reached sooner or later.

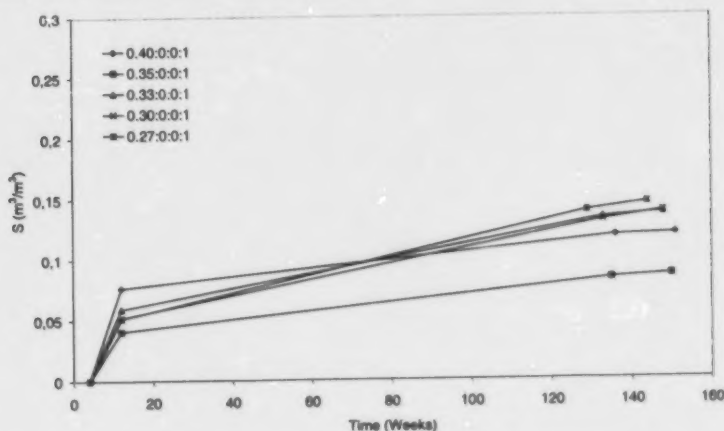


Fig. 4: Increase in degree of saturation for specimens without air-entrainment

3.4 Long-term water absorption in specimens with air-entrainment

These specimens have been treated in the same way as the specimen without air-entrainment with two exceptions; concretes 0.33:0:4:1 and 0.33:0:9:1 have only been subjected to three freeze/thaw tests. What can be clearly seen is that the degree of saturation is higher for all of the specimens with air-entrainment compared to those without air-entrainment (Fig 5). This is an indication that water absorption also occurs in the air pores. This is very dangerous for the frost resistance of the concrete.

3.5 Expansions during freezing

At four times during the three year period since casting, the specimens have been subjected to single cycles of freezing and thawing. The cycle is presented in Fig 6. The specimens were tested at the age 1, 3, 18 and 30 months. During the four freeze/thaw tests, the temperature and the length changes were measured. The temperature was recorded by a thermocouple in the centre of the specimen. A displacement transducer recorded the length changes. It was fixed on the invar-frame in which the specimen was placed. The maximum expansion is measured as the maximum deviation of the measured dilation from the extrapolated curve of linear thermal contraction. In (Fig 7), this is presented for specimens without air-entrainment. The expansion increased with increasing water-storage time.

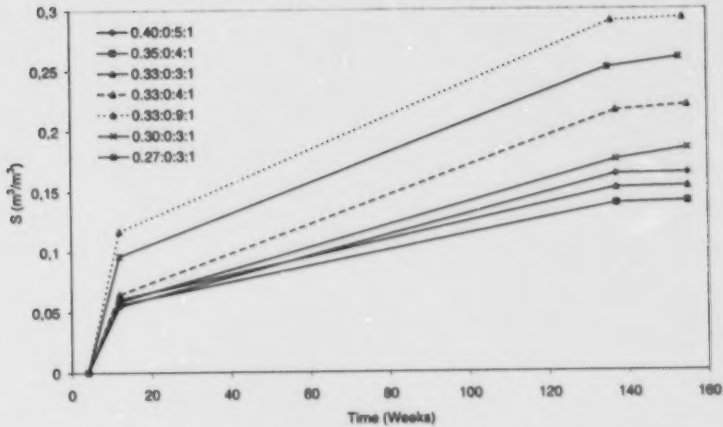


Fig. 5: Increase in degree of saturation for specimens with air-entrainment

Concrete with air-entrainment shows smaller expansions (Fig 8). The reason probably is that a large portion of the air-pore system stays air-filled after long-time water absorption. The large absorption in concretes with air-entrainment is therefore not big enough to cause damage. After very long time of water storage, however, even concrete with air-entrainment might become vulnerable to frost.

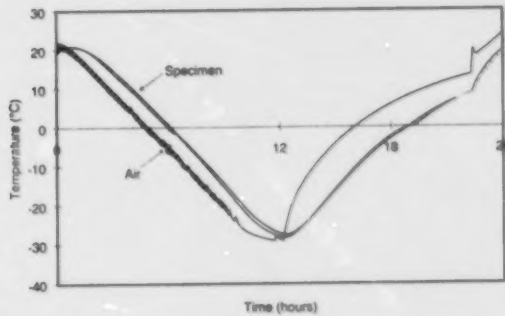


Fig. 6: Temperatures during freeze/thaw tests

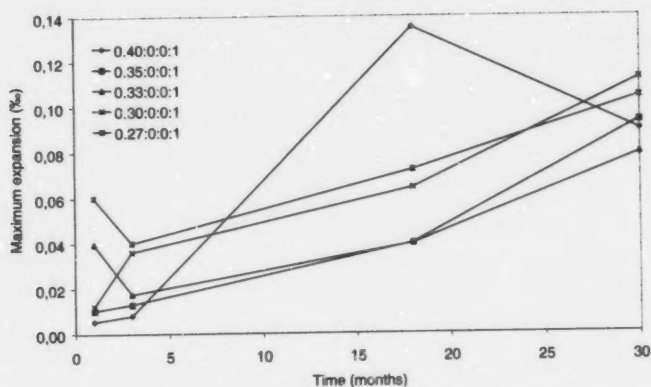


Fig. 7: Maximum expansion in specimens without air-entrainment

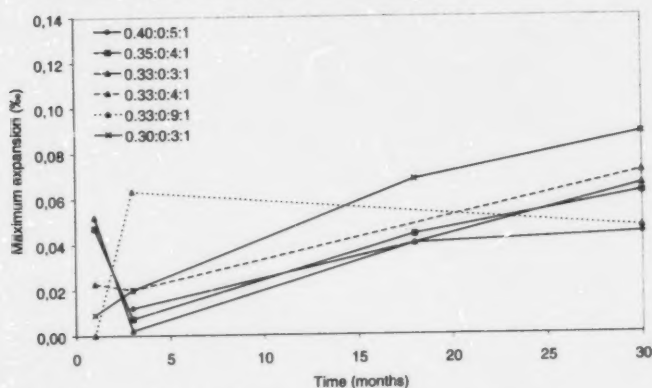


Fig. 8: Maximum expansion in specimens with air-entrainment

4 Conclusions

The effect of long time water storage on the frost resistance of high-performance concrete has been studied in this investigation. There are clear indications that self-desiccated pores might become re-saturated by long-term water storage and that air pores might absorb water. This emphasises the consideration of the long-term water absorption in self-desiccated concrete and air pores when designing freeze-thaw tests.

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21 ASSESSMENT AND INTERPRETATION OF IN-SITU STRENGTH OF CONCRETE

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Abstract

A series of penetration resistance tests (PRT) were conducted in the laboratory on concrete specimens to determine the in-situ characteristic strength of concrete. This was carried out in conjunction with core tests and cube compressive tests. It was found that the PRT and core tests tend to provide respectively an upper and lower bound estimate of the cube compressive strength. Major influencing factors including coarse aggregate content, proximity to reinforcement and carbonation that affect these two tests are reviewed and a method for the selection of an appropriate bias factor for each of the tests is proposed based on experimental studies carried out.

Keywords: appraisal, concrete, core tests, in-situ strength, non-destructive testing, penetration, retrofit, ultrasonic pulse velocity.

1 Introduction

Maintenance and upgrading of old RC structures are necessary for various reasons. Before any remedial action could be taken structural appraisal is essential. Such appraisal usually involve a process in which a structure is assessed by carrying out various tests to evaluate its existing condition to provide a basis for the recommendation of appropriate remedial measures to restore structural integrity, if necessary. NDT methods are usually fast and reasonably reliable. Hence, they are commonly used for structural assessment in conjunction with testing of cored concrete samples.

One method currently used in Singapore is the penetration resistance tests (PRT). This test (Bungey 1982, 1989) involves driving a 75mm specially designed steel probe into the concrete with a standardised explosive cartridge, and the standards are set out in (ASTM C803 1996). The principal physical limitations of this method are the need for adequate edge distance and probe spacing as well as the requirement that a member thickness of at least twice the anticipated

penetration is required. The effect caused by the presence of reinforcement in close proximity to a probe being driven is uncertain, and a minimum clearance of 50mm between probes and reinforcing bars has been recommended (Bungey 1982, 1989). The coarse aggregate content present in the mix will also contribute to a change in resistance against penetration of the probe. It is possible to correlate the PRT to the compressive strength of concrete provided that appropriate consideration is given to the influencing factors. (Lee et al. 1989) correlated the PRT with the compressive strength of concrete of a number of concrete mixes in an earlier study.

A more direct method of obtaining an estimate of the in-situ compressive concrete strength of an existing structure is to drill cylindrical cores for testing in compression. Standards are available in most countries describing the procedures for trimming, testing and to a lesser extent interpretation of core strengths (BS 1881 Part 120 1983; ASTM C42 1994; ACI 318 1995). When assessing the in-situ strength of concrete by coring, the following factors, viz., reduction of strength due to coring process, effect of state of stress, shrinkage and sustained loading effects, and the difficulty in strength evaluation based usually on a very limited number of core samples, have to be considered.

Although the need for in-situ measurements is clear, some of the features and especially the limitations of available test methods may result in unreliable values being used in structural appraisal. The objective of this investigation was to study some of the factors affecting the penetration resistance and core tests on concrete specimens. In particular, the following influencing factors affecting penetration resistance tests, namely, effect of age, effect of coarse aggregate content, effect of proximity of reinforcing bars and effect of carbonation are investigated.

2 Experimental programme

2.1 Materials used

Ordinary Portland cement, river sand, crushed granite aggregates of sizes ranging from 0 to 20mm were used. Steel bars consisted of high yield steel ($f_y = 460$ MPa) and links consisted of hot rolled mild steel ($f_y = 250$ MPa) were used in the reinforced concrete specimens. Four mix proportions were chosen (Mix 1, Mix 2, Mix 3 and Mix 4), consisting of cement: fine aggregates: coarse aggregates: water, viz., 1.0:2.5:2.66:0.65, 1.0:2.50:3.16:0.65, 1.0:2.50:3.72:0.65 and 1.0:3.22:3.22:0.70.

2.2 Test specimens

The specimens were divided into six series (A to F). In Series A, three mix proportions were used to study the effect of coarse aggregate content on PRT measurements. For each mix, four 300mm cuboid blocks were cast, accompanied by twelve 150mm cubes. Two methods of discharging the probe in the low-power range, were also investigated. The two discharging methods I and II were conducted on the specimens at ages of 3 and 28 days. In Method I, a standard adaptor is cushioned by 50mm air-gap from the charge, while, Method II uses another adaptor in direct contact with the charge as described (Ngui 1992). Coring was not carried out in this series.

Series B and C were conducted to study the effect of size and proximity of reinforcement on PRT. In addition to penetration resistance tests, core tests were

carried out on the same specimens to establish a relationship between the tests. A mix containing a coarse aggregate content of 0.42% by volume, was used to cast both the series. Series B comprises 4 specimens, 600mm long and 300mm in cross-section. Series C consists of two specimens, 700mm long and 400mm in cross-section. The specimens were subjected to PRT (using Method I low-power technique) and cores drilled and tested at 3 days. UPV measurements and PRT were taken at the same location to check for consistency of pulse velocities.

Series D and E using a coarse aggregate content of 0.42% by volume, were used to study the effect of proximity and neighbouring bars on PRT measurements. Tests carried out were similar to those on Series B and C, at 28 days using Method II in the low-power range of PRT. Tests were also carried out at 21 days using Method II on specimens in Series D.

In Series F, the effect of carbonation on PRT was studied. Three cubes of size 200mm were used. The blocks were placed in an environmental chamber with the environment within maintained at a relative humidity and temperature of 70% and 20°C, respectively. The concentration of carbon dioxide gas present in the chamber was maintained at 7% by volume. Accompanying specimens were also left at ambient conditions in the laboratory (28°C and 82% R.H). All the specimens were tested at 63 days. The specimens were weighed and UPV measurements were taken across the PRT locations prior to testing to ensure consistency.

2.3 Specimen preparation

Six cubes were cast for each mix to determine the compressive strength of concrete. Two 300mm size cuboid blocks were also cast for each mix to measure the penetration resistance of plain concrete. In each block, a 100mm diameter core was drilled vertically with respect to the casting surface right through the block. The core was then trimmed to obtain two cores, each measuring 100mm in length. The specimens were demoulded after one day and air cured in the laboratory till the age of testing. All specimens were tested dry at 3 and 28 days. In Series D, an additional block was cast to conduct PRT test at 21 days.

Cubes, cylinders and core samples obtained from specimens and blocks, were weighed and subjected to UPV tests to check for consistency and for the presence of voids in the concrete before testing. UPV measurements were also taken across PRT locations on blocks and reinforced specimens.

3 Experimental results and discussion

The UPV measurements made on reinforced specimens, blocks, standard cubes and cores did not show any sign of major voids or cracks. Typical average pulse velocity readings for Mix 1 and 3 at 3 and 28 days ranged from 3.37 to 3.69 km/s and 3.75 to 4.01 km/s respectively. The difference between the maximum and minimum depth of penetration of probes in each series of tests on blocks, cylinders and specimens averaged about 3mm. Thus, reflecting the consistency of the results obtained using the penetration resistance tests. Generally, tests carried out 3 days after casting indicated a greater variation in probe penetration than those carried out at 28 days. However, the difference is not significant enough to affect the overall results.

3.1 Penetration resistance test results

3.1.1 Effect of coarse aggregate content

The influence of coarse aggregate (CA) content on penetration resistance test is more significant in young concrete. From Fig. 1, it is seen that by decreasing the coarse aggregate content, the depth of probe penetration is increased. For a given method, at 28 days, the different amounts of coarse aggregate in the three mixes did not cause any significant change in the estimated cube strength obtained by PRT. This could be due to an improved bond at the aggregate matrix interface. Fig. 1 also shows that by using PRT to estimate the strength of concrete at a younger age, corrections have to be made especially in Method II for lower CA content. The increase in depth of penetration of the probe is probably due to the less developed bond at the aggregate/matrix interface at a younger age. A decrease in the energy of the probe as it leaves the driver is evident when Method I is employed. The absence of an air gap in Method II allowed the probe to be driven further into the concrete.

3.1.2 Effect of proximity of reinforcement

Fig. 2 shows the effect of proximity of reinforcement on PRT. A minimum spacing of 175mm has been suggested (ASTM C803 1996) between probes and hence, the zone of influence of the probe being driven into a concrete mass is thus expected to radiate a distance 87.5mm from the centre of the probe. (Nasser and Al-Manaseer 1987) reported that as long as the nearest steel bars in concrete are located outside this zone of influence, the presence of steel bars in concrete did not affect the probe penetration. This is also reflected in Fig. 2 which indicates that probes driven at a distance of 75mm or more from the nearest reinforcement bar tend to provide a similar estimate of the cube strength. In general, overestimation of the concrete strength was evident by PRT when probes were driven 90mm or less and 50mm or less from the nearest bar respectively, when specimens tested by Method I and Method II.

3.1.3 Effect of carbonation

(Nasser and Al-Manaseer 1987) reported that there is no effect of carbonation on PRT. However, the limited number of penetration resistance tests conducted on 200mm cubical blocks indicate a significant decrease in probe penetration as the depth of carbonation increases (Fig. 3). This could be due to the fact that carbonation affects the surface zones of the concrete only. If the depth of carbonation is significant compared to the penetration depth then the penetration depth of the probe decreases.

3.1.4 Velocity tests on probes

Some results on the actual velocity of the probes as it leaves the driver is summarised briefly (Tan 1996). Two different Windsor probe driver units were used and the velocity of the probes measured were reported in Table 1. From Table 1, it is observed that there is no significant difference in the average velocities of probes discharged by the two driver units although they were purchased ten years apart. The first unit is of a model not in current production and has been in use for about ten years. Method A used in measuring the discharge velocity of the probe, the two driver units have the same average discharge velocity. However, the new driver exhibited a

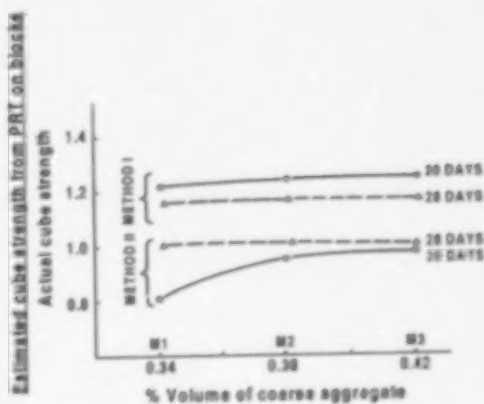


Fig. 1: Effect of volume of coarse aggregate on PRT

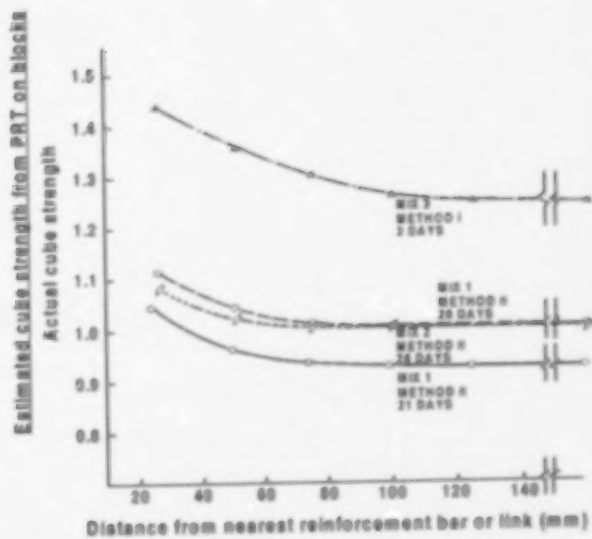


Fig. 2: Effect of proximity of reinforcement on PRT

slightly higher coefficient of variation when compared to the old driver. It is also observed that the average probe velocity measured ranged between 180 m/s and 187 m/s, which more or less confirms with the manufacturer's claim that this average is 183m/s. However, actual velocities measured can be as low as 166.7 m/s or as high as 201.7 m/s.

Table 1: Velocities of the probe

Type of experiments	Average velocity (m/s)	Coefficient of variation (%)
Method A - old driver (Tan 1996)	185.0	2.6
Method B - new driver (Tan 1996)	185.0	3.0
Method C - new driver (Tan 1994)	179.8	3.4
Method D - new driver (Lee 1995)	187.0	3.7

3.2 Core test results

The average compressive strength of cores drilled from corresponding specimens cast using the same mix were generally 10 to 40% lower than that obtained by crushing 150mm cubes. This could be caused by some of the factors discussed earlier namely the limited number of core samples and the damage caused during coring process.

3.2.1 Effect of coarse aggregate content

Fig. 4 shows that the coarse aggregate (CA) content of concrete block specimens from which cores were extracted affects the ratio of the estimated cube strength to the actual cube strength at both 3 and 28 day strengths. The ratio is significantly low for all the mixes at 3 days. This may be due to weaker bond at the aggregate matrix interface at 3 days resulting in more damage caused by coring.

4 Analysis of test data

4.1 Interpretation of penetration resistance and core test results

The data obtained from core tests and penetration tests can be used to estimate the characteristic strength, taken for the present study to be mean minus 1.64 times standard deviation. Since, the factors contributing to the statistical variation in the core tests and penetration resistance tests are different, their combined results are not cohesive, even, when testing specimens cast using the same batch of concrete. Hence, there is a need to calculate the characteristic strength obtained from cores and that from probes independently. The actual characteristic strength f_k , may be derived from the individual characteristic strengths from cores and probes by multiplying the latter by bias factors for cores and PRT respectively. For the present study, these factors are assumed constant.

$$\gamma_c f_{ck} \approx f_k \quad (1)$$

$$\gamma_p f_{pk} \approx f_k \quad (2)$$

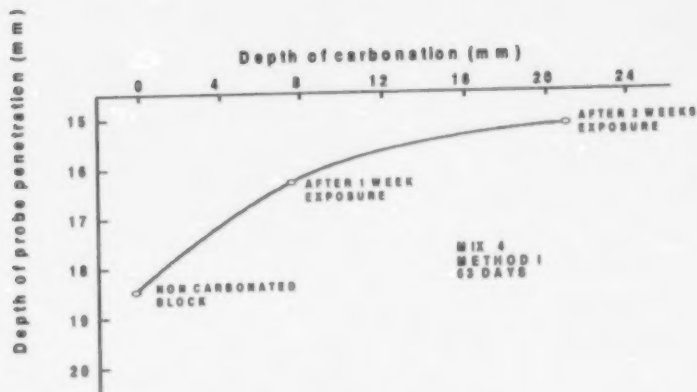


Fig. 3: Effect of carbonation on probe penetration

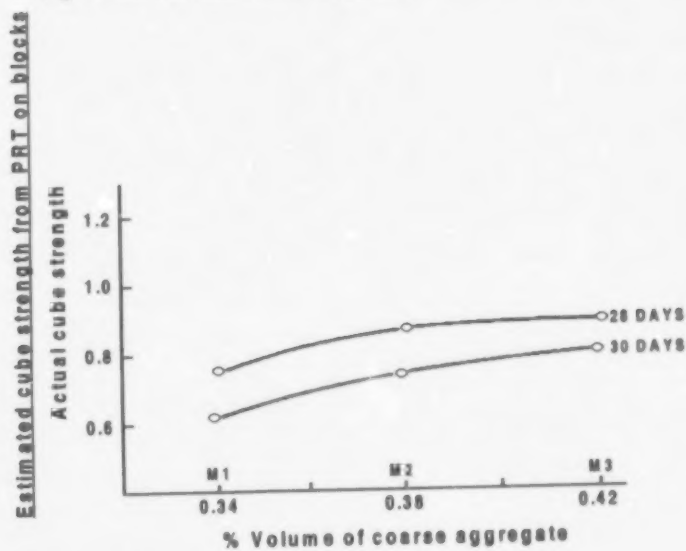


Fig. 4: Effect of volume of coarse aggregate on cores

Where, v_c and v_p are constant bias factors. If f_k is underestimated by the core results and overestimated by the probe results, then,

$$f_{ck} < f_k < f_{pk} \quad (3)$$

thus

$$1.0 < v_c < f_{pk}/f_{ck} \quad (4)$$

$$f_{ck}/f_{pk} < v_p < 1.0 \quad (5)$$

Bayesian statistical approach was employed to provide a range of bias factors for strength estimation, obtained from PRT and core test. The steps involved are as follows:

1. The characteristic strength from corresponding cubes was obtained and compared with the uncorrected characteristic strength from cores and PRT.
2. The maximum and minimum ranges of bias factors were also obtained. By predicting a range of bias factors limited by factors affecting both PRT and cores, an estimated range of characteristic cube strength is obtained. Since the characteristic strength obtained from cubes is known then the actual bias factors (assumed constant) for PRT and cores can be determined.

Bias factors and characteristic strength of specimens tested is reported in Table 2. All these values obtained using low-power method I are within the expected range of bias factors reported in an earlier study (Lee et. al. 1989). Observing the bias factors obtained using low-power method II, it is assumed that the bias factor for PRT should be less than 1.0.

Table 2: Bias factors and characteristic strength for specimens tested.

Test specimen	Method of driving	Age	v_c	v_p	f_k (MPa)
Mix 1	Low-power I	28 days	1.47	0.90	23.7
Mix 2	Low-power I	28 days	1.20	0.89	26.9
Mix 3	Low-power I	3 days	1.33	0.88	18.3
Mix 3	Low-power I	28 days	1.14	.087	27.8
Mix 1	Low-power II	28 days	1.29	0.99	20.3
Mix 2	Low-power II	28 days	1.35	1.00	27.3

5 Conclusions

The following conclusions can be drawn from this investigation:

1. An increase in coarse aggregate content increases the ratio of estimated cube compressive strength from PRT to actual cube strength.
2. The presence of reinforcement may affect the depth of probe penetration depending on the proximity of probe location to the nearest reinforcing bar. It was observed that the presence of reinforcing bars within the proximity of 50mm generally cause an overestimation of the in-situ cube compressive strength using PRT.
3. An increase in the depth of carbonation in concrete decreases the depth of probe penetration.

4. An increase in coarse aggregate content increases the ratio of estimated cube compressive strength from cores to actual cube strength.
5. The estimation of in-situ characteristic cube strength of concrete can be correlated with PRT and core test results using the Bayesian statistical approach. However, the nature of influencing factors need to be identified and the selection of the appropriate overall bias factor for each situation need to be systematically explored so that specific guidelines may be proposed to aid engineers engaged in this field of work.

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22 INVESTIGATION AND DIAGNOSIS OF REINFORCED CONCRETE STRUCTURES IN AGGRESSIVE ENVIRONMENTS

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Abstract

The paper presents some considerations about the investigation and diagnosis activities of reinforced concrete structures placed in aggressive environments, including ones which were subjected to previous intervention. There are presented as the main steps of the investigation of the state of the members/structure, and those which lead to establishing the diagnosis. In the end, the diagnosis gives an estimation of the damages suffered by the structure and the measures, necessary for its rehabilitation, in accordance with the nature and degree of the existing damage. The paper synthesizes prolonged experience of the author in this field, finalized by the elaboration of National specific technical regulation.

Keywords: aggressive environments, diagnostic, intervention measures, investigation, reinforced concrete structures

1 Introduction

During the last decades, many reinforced concrete structures, especially the ones sheltering technological processes of a chemical nature, have suffered, significant deteriorations, sometimes severe ones, after a relatively short service time, as consequence of the action of aggressive environments.

The appearance and fast evolution of the damages generated by corrosion have imposed the necessity to determine their implications on the durability and service safety of the structure, in order to take the necessary measures of intervention.

The practice of a systematic investigation of the structure placed in aggressive environments permits both the detection of dangerous situations of damage, and the decision to undertake immediate measures of intervention with

the aim to prevent casualties, important material losses, technological accidents or undesired ecological effects, as well as to stop accelerated deterioration processes on the members/structures.

When making a diagnosis concerning intervention measures for the rehabilitation of the structures exposed to aggressive environments, one must consider not only the effect on the building materials (concrete, reinforcement), but also the on the broader issues of design and execution of intervention works for different categories of structures. The following concepts in this respect are worth mentioning:

- the construction has to be regarded and analysed as a whole, taking into account its initial design, as well as the evolution of its condition up to the moment of intervention;
- the causes of damage due to corrosion are multiple and have to be correctly and completely identified; one must also consider causes other than corrosion which damaged the structure, as well as the permanent danger of seismic actions;
- the damage generated by the action of aggressive environments occur, as a rule, on limited parts of the elements/construction, but the intervention solution must concern the construction as a whole; some local damages induced by corrosion may lead to altering the mechanisms of load distribution considered in the initial design;
- the investigation/diagnosis and design activities are mutually interconnected and intrinsically interdependent.

2 Investigation

The investigation on the state of the reinforced concrete members/structures placed in aggressive environments must comprise the following main steps:

- the analysis of the technical documentation of the construction;
- the appreciation of the service conditions of the construction;
- the actual investigation of the damage state of the members/structure: directly on the members (in situ) or on samples taken from the members (in laboratory).

The aim of the analysis of the technical documentation is to know the biography of the structure/members, concerning: the description of the structure/members; design data; execution data; information about the service life of the structure (including the intervention measures to which it was subjected). This data is provided by the design engineer and the owner/user of the construction, is included in the materials specifications, quality certificates and the documents concluded between the design engineer, builder and owner/user, and is included in the Technical Book of the construction.

The service conditions of the construction are estimated by taking into account the characteristics of the aggressive environment (indoor and outdoor), the technological process going on inside the construction and the function of the technological installations, including the exhaust equipment (ventilation, etc.).

The investigation of the damage state of the members/structure in service are made by means of the following investigation methods:

- the visual examination;
- the use of non-destructive testing methods;
- the use of methods that require the taking of samples, but which do not endanger the service safety of the members/structure.

The visual examination of the state of the structure/members, and of the anticorrosive protection applied on their surfaces, includes: the appreciation of aspect changes of the concrete and the (apparent) reinforcement surfaces; the presence of defects/degradation due to corrosion; the evaluation of the changes in condition of the anticorrosive protection. The results of the visual examination are registered in the form of surveys of defects/degradations on the lay-outs and sections of the construction, with details on the investigated members, indicating their position and the extent of the damage, in order to provide their identification on the structure. With this purpose, for each type of defect or damage, will be adopted a conventional mark. Any time possible, pictures of the defects or damages will be taken.

Non-destructive methods of investigation are used for the control of the size of the members, the establishment of certain characteristics of the concrete (strength, density, module of elasticity, a.o.), verifying the position of the reinforcement and/or the presence of metallic pieces, embedded in the concrete, detection of hidden defects in the concrete or the reinforcement (segregations, cracks, etc.), and for determining the condition and risk of corrosion of the reinforcement embedded into concrete (by electrochemical methods).

The methods of investigation using concrete samples - under condition that they do not endanger the service safety of the members - include the measurement of the concrete cover, the appreciation of the depth of the concrete that suffered changes under the action of corrosive agents, the examination of the state of the reinforcement, the determination of the type of damage induced by corrosion, as well as for performing laboratory tests. Under circumstances of extreme necessity (for instance in presence of stress corrosion cracking), samples are taken of the reinforcement itself, provided that measures to protect the integrity and the service safety of the investigated member/construction are taken.

The tests and determinations performed in laboratory concern the physico-chemical, electro-chemical and physico-mechanical tests, performed on the concrete and reinforcement samples taken off from the structures and consist mainly of: the measurement of the mechanical characteristics, the humidity, the water absorption and the porosity of the concrete; the determination of the alkalinity of the concrete (pH of the aqueous solution) and the concentration of corrosive ions (Cl^- , SO_4^{2-} , a.o.) in the concrete mass; measurement of the mechanical and physico-chemical characteristics of the reinforcement steel (composition, metallographic examination, a.o.).

The corrosion state of the reinforcement steel is evaluated in accordance with the following main parameters: the characteristic of corrosion (general, uniform, local, pitting, stress corrosion cracking etc); the nature, structure and thickness of corrosion products; the physico-mechanical characteristics of the reinforcement; the structure of the steel. In the case of diagnosing general and

local corrosion phenomena of the reinforcement steel, the use of a conventional corrosion degree estimation system is recommended.

We draw attention to the phenomenon of stress corrosion cracking of high strength steels, in which case complex investigations become necessary, with the aim to determine the chemical composition, the purity, the microstructure of the steel, as well as the presence of intergranular/transgranular cracks, of defects or local modifications of structure, etc. Such investigations are performed by specialized laboratories, using specific metallographic analysis methods, X -rays diffraction, electronic diffraction, electro-chemical and acoustic methods, etc.

3 Diagnosis

The diagnosis is the result of investigation (the complexity of which depends on the degree of damage to the members/structure) and represents the basis for adopting intervention measures.

The diagnosis concerns first evaluation of the damage state induced by corrosion, specific to the analysed environments, and second, the initial defects and the degradations induced by other causes (earthquakes, fire, shocks, landslides, etc.).

The main steps which lead to the establishment of a diagnosis are:

- The preliminary examination of the construction and its project, consisting mainly of the visual analysis of the bearing members of the construction and the engineer evaluation of their state in the context of the knowledge of their general structural construction. As result of the preliminary examination conclusions can be drawn up for temporary measures, in order to prevent imminent or potential accidents/breakdowns.

The detailed investigation by means of measurements and both "in situ" and laboratory tests, aimed to point out: (initial) defects and damages, other than the ones induced by corrosion, with an explanation of their causes; defects of the technological installations, which are one of the major causes of corrosion induced degradations; other defects, on ventilation, exhaust and waste water sewage equipment, waterproofing, a.o.

- Data processing and analysis of the safety degree of the construction by interpreting the data, estimation of loads on the service time of the construction, a.o.
- The establishment of the diagnosis and proposals concerning intervention measures.

All the information above concerning the investigation, analysis and diagnosis will be centralized in a written report drawn up by the specialists/experts who performed these activities.

4 Intervention measures

The intervention measures on the bearing structure of the construction are proposed in correlation to the nature and degree of the damages, presented by the members of the investigated construction:

- for damages that do not affect the safety and the stability of the construction, the measures consist of interventions for repairing/maintaining the anticorrosive protection;
- for damages which affect the durability of certain members, without imminently endangering the resistance and stability of the construction, the measures consist of interventions for repairing the members and providing the necessary anticorrosive protection;
- for damages which may dangerously affect the resistance and stability, the measures consist either on entirely or partially disaffecting and demolishing the construction or in interventions for repairing/strengthening its bearing structure and providing subsequently the necessary anticorrosive protection measures.

Taking into account the particular character of the investigation, diagnosis and of some intervention measures, it is recommended that these activities would be performed by pluri-disciplinary teams of corrosion and bearing structures specialists or with specialists in other fields (according to the case).

Based of the research results, and the author's 30 years of experience in this field, national technical regulations stipulating the conditions and the ability to perform investigation and diagnosis on reinforced concrete structures placed in aggressive environments have been achieved.

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23 GAS PERMEABILITY OF CONCRETE: DEFINITION OF A PRECONDITIONING PROCEDURE FOR MEASUREMENTS AND CROSSOVER TRIALS

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Abstract

This paper describes the main results obtained jointly by a number of French laboratories within the framework of a group under AFREM (Association Française de Recherches et Essais sur les Matériaux et les Constructions).

The objectives of this group were to define a preconditioning process which would result in a better distinction between the various types of concrete with an acceptable test length, and to draft a recommendation for measuring the gas permeability of concrete.

The procedure for measuring the gas permeability of concrete provides good reproducibility. However, the equipment used does not allow the measurement of permeability values greater than 10^{-19} m².

A conditioning procedure for predrying a test sample before measurement was defined. This conditioning procedure is the result of a compromise among the necessary attributes of a test, which should be easy to apply and as rapid as possible, while not degrading the test sample excessively. A drying temperature of 80°C was selected, with two permeability measurements (at seven and 28 days).

It was noted during the tests that the moisture content of the material had a considerable impact on gas transfer, and consequently that measuring permeability alone was not sufficient to distinguish between concrete types. It is absolutely essential that this test be accompanied by a measurement of mass loss, and a graph representation of permeability as a function of mass loss seems to be the only effective way of distinguishing between types of concrete.

Keywords: permeability, concrete, drying, measurements, mass loss, saturation

1 Introduction

The permeability of concrete to gases is one of its basic durability indicators. Accordingly, permeability measurements must follow a well-defined procedure to produce values which can be considered reliable.

The "permeability" subgroup of the AFREM "concrete durability" group has therefore worked to establish a procedure for conditioning concrete before measurements are taken, in order to better distinguish among types of concrete within an acceptable period of time (Quenard and Carcasses, 1997).

The moisture content of the material has a considerable influence on gas transfer and was taken into account.

2 Materials and equipment

2.1 Materials

Two concrete formulations were studied in four trials: a high-performance concrete (HPC) and a plain concrete (B40), the properties of which are given in Table 1. The concrete pourings were kept under static conditions for a day at 20°C, then removed from their moulds and kept in a humid room (20°C, 95% RH) until old enough for testing to begin, or 28 days. Between the 14th and 28th days, core samples were taken and sawn to obtain disks 15 cm in diameter and 5 cm high.

Before the start of testing, the side of each sample was covered with an aluminum cling film to achieve unidirectional drying.

Table 1: Composition of the two concrete types in the crossover trials

Composition	HPC	B40
cement	400 kg	400 kg
type of cement	CEMIII 52.5	CEMI 52.5
sand 0/5	660 kg	844 kg
fine gravel 5/12.5	415 kg	1153 kg
gravel 12.5/20	865 kg	/
plasticizer	20 kg	/
water	125 kg	196 kg
slump	17 cm	
S28d	75 MPa	41 MPa
W/C	0.31	0.65
porosity	8.5%	14.6%

2.2 Porosity, mass loss, water content and degree of saturation

2.2.1 Porosity

The porosity ϵ is defined by the following equation:

$$\epsilon = \frac{M_{\text{ctd}} - M_{\text{dry}}}{M_{\text{air}} - M_{\text{wr}}}$$

where M_{ctd} = mass in air of the saturated, coated test piece
 M_{air} = initial mass in air of the saturated test piece
 M_{wr} = initial mass in water of the saturated test piece
 M_{dry} = mass of the coated test piece dried at 105°C

2.2.2 Mass loss

The mass loss ΔM , for a measurement time X in days, is calculated using the following equation:

$$\Delta M = \frac{M_{\text{ctd}} - M_X}{M_{\text{ctd}}}$$

where M_X = mass of the test piece for a measurement time X .

2.2.3 Degree of saturation

The degree of saturation S is calculated using the following equation:

$$S = \frac{M_X - M_{\text{dry}}}{M_{\text{ctd}} - M_{\text{dry}}}$$

During the different tests, the dry mass M_{dry} was not always measured. Nevertheless, considering the low variability of porosity (Table 1), M_{dry} can be estimated from the test sample volume V_i fixed by the measurement method (diameter of 15 cm, thickness of 5 cm).

2.3 Equipment

The instrument used was a constant-head permeameter of the CEMBUREAU type (Fig. 1) with a range of absolute pressures from 1 to $6 \cdot 10^5$ Pa (1 to 6 bars).

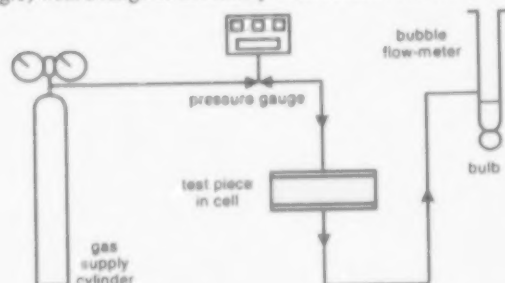


Fig. 1: Flow diagram of permeameter operation

3 Tests and results

3.1 Preconditioning of HPC at 50°C, 80°C and 105°C

When the concrete is 28 days old, preconditioning starts with drying in a ventilated oven (at different temperatures depending on the trial). When drying is completed, the test sample is kept for one day in a sealed bag at 20°C to allow it to return to thermal equilibrium. The mass losses of the samples are monitored at each stage of preconditioning.

At the end of preconditioning, the gas permeability is measured. The samples are then returned to the oven for another period of drying.

Three trials corresponding to three drying temperatures (50, 80 and 105°C) were carried out with the HPC described in Table I.

To begin with, drying in an oven regulated to 50°C, 80°C and 105°C in turn, in a laboratory regulated to 23°C and 50% RH, results in corresponding relative humidity values surrounding the test sample of approximately 11%, 8% and 3%. Thus the drying conditions are very severe.

The drying curves in Figure 2 show the good reproducibility obtained among the laboratories for drying at 50°C for three months. These curves are more scattered for drying at 80°C and 105°C.

The drying kinetics increase with temperature, as temperature plays its usual role as a kinetics accelerator. Equilibrium thus seems to be reached quite quickly (between 10 and 20 days) when drying is performed at 105°C, while at 80°C and 50°C the drying curves are still changing substantially after 50 days of drying.

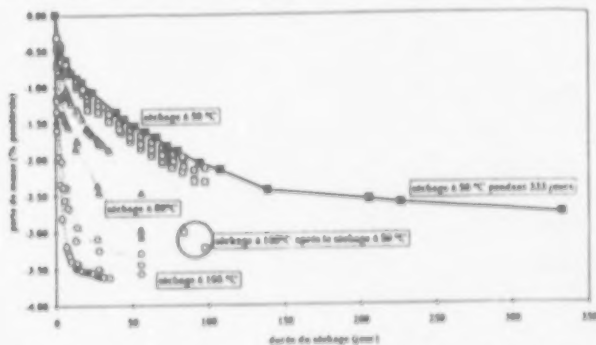


Fig. 2: Weight loss as a function of duration and temperature of drying

It is also interesting to note that drying at 50°C, continued over a very long period of about one year, results in a mass loss approaching the value obtained at 105°C. This development is explained by the fact that, for the three drying temperatures (50, 80 and 105°C), the surrounding relative humidity is very low (between 3% and 10%) and the drying power of the air is almost identical in all three cases.

Another observation is that, when certain samples are dried at 105°C after drying at 50°C, their mass losses tend toward those obtained by an initial drying at 105°C.

During the drying process, moreover, the distribution of water through the material is not homogeneous: there is a water content gradient. The moisture content profile was monitored by gammametry during drying (Fig. 3). This gradient was not disregarded but, since a homogeneous redistribution of the water in the sample could not be expected within a reasonable time, the permeability measurements were taken with a non-homogeneous distribution of moisture and the average saturation level was considered representative of the condition of the material.

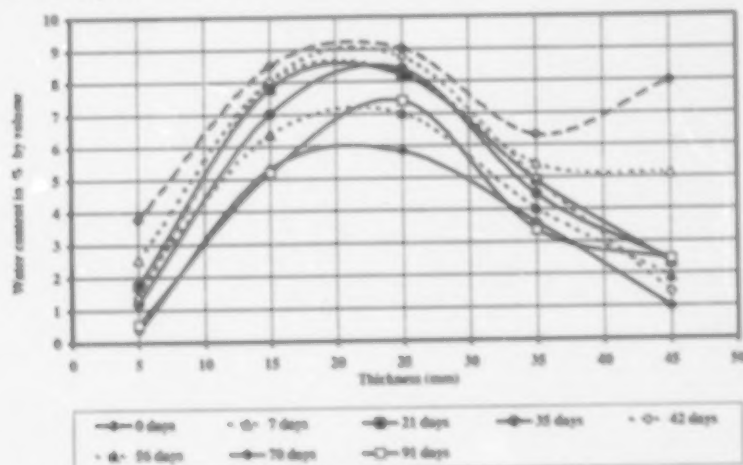


Fig. 3: Water content profiles for different drying times at 50°C

3.2 Permeability of the HPC

At 50°C, the permeability was measured every seven days for three months. In addition, for the sample on which drying was extended to approximately one year, the permeability was also measured during that period.

For conditioning at 50°C, the permeability was measured with a gauge pressure of 5 bars, while for 80°C and 105°C this pressure was no more than 1 bar.

The results for permeability as a function of drying time and degree of saturation of the samples are given in Figure 4. It can be seen that the permeability varies markedly with drying time. It increases by about two orders of magnitude (from 10^{-18} to 10^{-17} m²) between 20 et 100 days of drying, or for a reduction in saturation from 70 à 30%. Moreover, the permeability values measured after drying the samples at 105°C increase by yet another order of magnitude. Finally, continuing the measurement of permeability for drying times of about 300 days at 50°C shows that the permeability tends toward the value obtained by drying at 105°C (Fig. 4).

Figure 4 shows that scattering is greater for the permeability measurements than for the drying curves, and that it increases with shorter drying times (less than 50 days) (Fig. 4). This suggests that there is a threshold value for saturation below which it is impossible to measure any gas flow with sufficient accuracy: there is no percolation path allowing the flow of gas. In this zone, a small variation in mass loss can result in a pronounced variation in permeability.

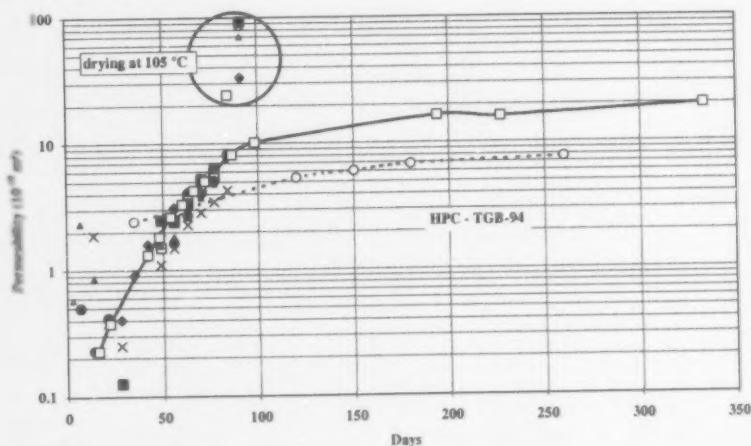


Fig. 4: Permeability as a function of drying time at 50°C

A series of values obtained during an earlier study on concrete prepared with an identical formulation (curve marked "HPC-TGB94") has been included in Figure 4. Although these values are lower, they show the same trend as a function of drying time.

3.3 Preconditioning of the HPC and B40 at 80°C

In the final trial, the two concrete formulations described in Table 1 were tested.

The B40 lost more water (Fig. 5), as its porosity is greater (Table 1). In addition, its mass loss was more rapid, as the pores in this concrete are definitely larger in size.

When permeability is shown as a function of the degree of saturation of the material (Fig. 6), it is apparent that, for high saturation levels, the measurements for the HPC are very scattered, with a maximum difference greater than one order of magnitude (Fig. 6). This scattering may also be related to a percolation effect: a small variation in water content can result in a significant change in the connectivity of the porous network. Moreover, it is very important to mention that this representation does not allow us to distinguish between the two types of concrete.

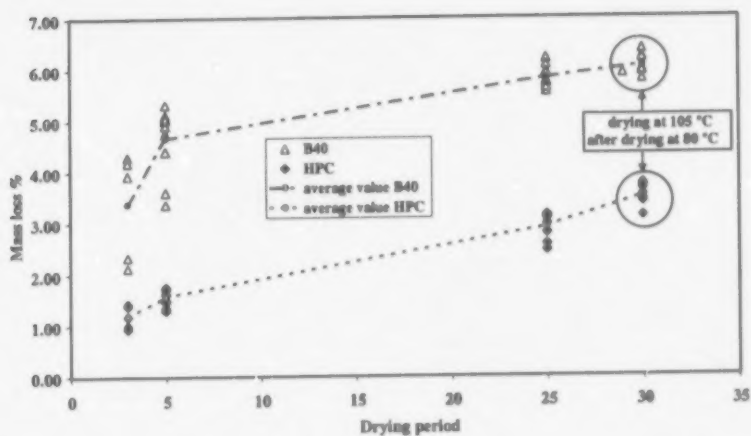


Fig. 5: Mass loss as a function of drying time

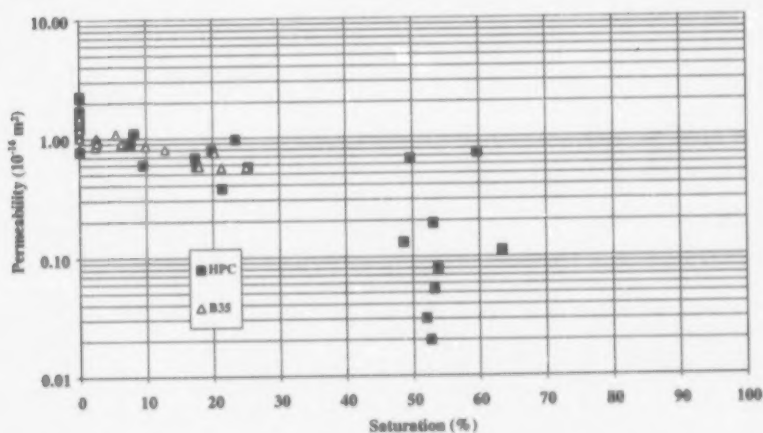


Fig. 6: Permeability as a function of degree of saturation - 4th trial

Comment: In this section, permeability values are expressed in 10^{-16} m^2 and no longer in 10^{-18} m^2 as in the preceding sections.

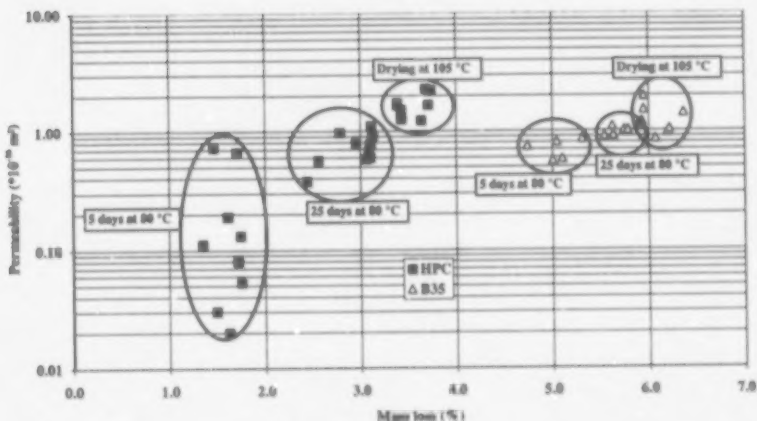


Fig. 7: Permeability as a function of weight loss: HPC and B40.

This is why, in Figure 7, the change in permeability is plotted as a function of mass loss: this type of representation allows us to differentiate between the two types of concrete studied.

In fact, for a given saturation level, the HPC and B40 show similar permeability values, but a given saturation level does not correspond to identical mass losses.

At this point, it is important to point out that plain concrete (PC) which is close to its water saturation point can have a gas permeability value lower than HPC in the "dry" state.

Figure 8 shows most of the permeability values for the HPC obtained during the four trials. We see that the points are all clustered in a beam showing a variation of about three orders of magnitude between the "dry" state and a saturation level close to 80%. It is interesting to compare this tendency to that obtained by C. Gallé (Gallé, 1997) for cement pastes. Finally, Garboczi and Bentz (Garboczi and Bentz, 1991) obtained a percolation threshold of 18% for porosity using a simulation model for the hydration of cement.

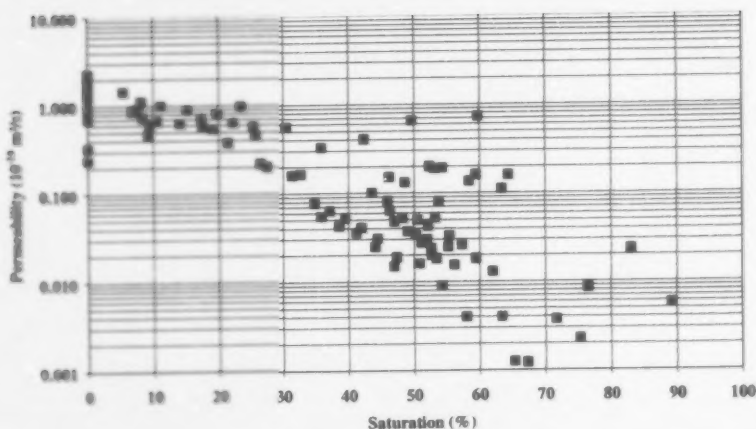


Fig. 8: Permeability as a function of degree of saturation for the HPC

4 Conclusion

The work by the permeability subgroup enabled it to draft a recommendation for the measurement of gas permeability. An essential element of this recommendation is the definition of a conditioning procedure for predrying test samples before measurements are taken. This conditioning procedure is the result of a compromise among the necessary attributes of a test, which should be easy to apply and as rapid as possible, while not degrading the test sample excessively. A drying temperature of 80°C was selected, with measurements at two points in time (seven and 28 days).

Moreover, measuring permeability alone is not sufficient to distinguish between concrete types; it is absolutely essential that this test be accompanied by a measurement of porosity to calculate the degree of saturation and loss of mass. The representation of permeability as a function of mass loss seems to be the only effective way of distinguishing between types of concrete.

5 Acknowledgements

The authors would like to thank all the partners in the "permeability" working group for their effective participation:

- CERIB: Ms C. Dubois, Mr R. Bodet
- CTG (Ciments français - Italcementi): Ms S. Morgano and Ms B. Bourdette, Mr F. Demars.
- EDF - CEMETE: Mr A. Ponsonnet.
- CEBTP: Mr Tran.

- LCPC: Ms V. Baroghel-Bouny.
- LERM: Mr H. Homain; Mr B. Quénée, Mr Rajnak
- LMDC: Prof. J.P Ollivier, Ms N. Ruscassier
- CSTB: Mr P. Kalifa, Mr H. Sallée and Ms M. Tsimbrovska

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24 SO₂ ENVIRONMENT DEGRADATION OF SYNTHETIC PASTES OF PORTLAND CEMENT

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Abstract

The interaction between synthetic pastes of hydrated phases of Portland cement and the SO₂ in a controlled atmosphere has been studied. Tests were carried out in a simulation chamber in the following work conditions: 0.3 ppm SO₂ concentration, temperature 25 °C and 95% relative humidity for 6 and 12 months.

The results obtained show that the sulphation process of pastes occurs in competition with the carbonation process, depending on the mineralogical and microstructural characteristics of the pastes.

Keywords: cement paste, calcium silicate, calcium aluminate, environment degradation, sulphation, chamber test, carbonation.

1 Introduction

The project "Environmental Deterioration of Ancient and Modern Hydraulic Mortars (EDAMM)" has identified the problem of durability on old hydraulic mortars in the built heritage. In addition, restoration works on buildings during this and the previous century have been performed with hydraulic mortars using cement and hydraulic lime as a binder (ICCROM, 1981). These mortars are currently suffering degradation (Zappia et al., 1994). Thus, among other objectives, the EDAMM project aims at understanding the decay mechanisms due to environmental action in hydraulic mortars used in historic masonry. The project also attempts to improve the scientific understanding of the parameters of mortars responsible for the type and speed of degradation due to atmospheric deposition. With this in mind, the sulphation process was simulated on different types of hydraulic mortars in a laboratory chamber.

The present contribution focuses on the environmental sulphation process studied on paste specimens of synthetic hydrated Portland cement (HOPC) and its single components expressively prepared.

2 Experimental section

Portland cement components (C_3S , C_3A and C_4AF) were synthesized using pure chemical reagents. Stoichiometric amounts of the reagents were weighed and the mixtures ground and homogenised using an ethanol dispersive medium. The mixtures were heated at 1500 °C until the reactions were complete; the purity of the obtained products was confirmed by means of x-ray diffraction.

Five different binders were prepared using the synthetic compounds and mixed with water; the pastes were poured into 10x10x5 mm prismatic moulds. The experimental conditions during the preparation of pastes and curing of prisms are reported in Table 1. 20 prisms per binder were cured at the indicated conditions long enough to allow their complete hydration.

A characterisation of the specimens before the exposure tests was performed by means of microstructural, mineralogical and chemical analyses.

Simulation experiments were carried out in a flow chamber at controlled temperature, relative humidity and SO_2 concentration. The paste specimens of synthetic hydrated Portland cement (HOPC) and its single components were exposed for a period of 6 and 12 months in air with 0.3 ppm SO_2 as pollutant and 0.50 l min^{-1} flow gas velocity, at 25 °C temperature and 95% relative humidity.

To ensure that the sulphation reactions would occur only on the exposed surface of the samples, a passivation treatment was applied to the samples by means of a graphite paint on the 5 non-exposed faces.

After 6 and 12 months, the specimens were removed from the simulation chamber, cleaned by removing the passivating graphite paint and preserved in an inert atmosphere. Pore structure identification, the determination of SO_2 effects and mineralogical investigation were subsequently carried out by means of mercury intrusion porosimeter (MIP), specific surface (BET), ion chromatography (IC), X-ray diffraction (XRD), infrared spectroscopy (FTIR), thermal analyses (DTA-TGA) and electron microscopy (SEM-EDX and TEM).

3 Results and discussion

3.1 Pore structure

The evolution of the total porosity in the non-exposed and exposed specimens, measured by MIP, is presented in Table 2, alongside the BET specific surface.

In order to define the pore sizes, the evolution of the pore size distribution was studied as a function of time. The results are presented in Tables 3 and 4. For each type of paste, the pore volume between a lower and an upper diameter is reported for the following time intervals: before exposure (0m), after 6 months (6m) and after 12 months exposure (12m).

Table 1: Samples prepared for the exposure tests

Sample	Binder type	Reaction products	Water/ solid (wt.)	Curing conditions
P1	C ₃ S	C-S-H (I) + CH	0.45	21°C; 100% RH.
P2	C ₃ S + 25%SiO ₂ (gel)	C-S-H (I) + C-S-H (II)	0.77	21°C; 100% RH*
P3	C ₃ A	C ₃ AH ₆	0.6	21°C; 100% R.
P5	C ₄ AF	C ₃ (A,F)H ₆	0.3	21°C; 100% RH
P6	C ₃ S + 12%C ₃ A + 12%C ₄ AF + 6%CaSO ₄ ·2H ₂ O	C-S-H + CH + Ett	0.5	21°C; 100% RH

* first week at 40°C and 100% RH

Table 2: Total porosity and BET specific surface of the exposed and non exposed pastes

Analysis	Exp.time (months)	P1	P2	P3	P5	P6
Total porosity (cm ³ g ⁻¹)	0	0.094	0.294	0.241	0.062	0.111
	6	0.058	0.270	0.219	0.059	0.071
	12*	0.069	0.205	0.162	0.061	0.073
BET area (m ² g ⁻¹)	0	9.371	12.586	0.730**	2.969	7.953
	12	6.673	40.830	3.228	5.063	8.576

* Average of two values

** At 6 months: 2.535 m² g⁻¹

The general tendency is a decrease in the total porosity of the tested specimens after 12 months. Pastes P1, P2, P3 and P6 undergo a very conspicuous drop in porosity (approximately 27%, 30.5%, 33% and 34.5%, respectively). For paste P5, the decrease is limited to 1%.

Concerning paste P1, the MIP test showed a concentration of the pores in the range of 0.01-0.05 μm . During exposure, a progressive pore refinement was observed. This refinement was reflected in the decrease in total porosity, as well as in the total volume of pores in the 0.01-0.05 μm pore fraction. There is a very marked concentration of pores below 0.05 μm , whereas the volume of pores higher than this diameter is very small.

The much higher porosity of paste P2 can be explained by the higher water content of the sample. Moreover, the pore size distribution of pastes made with silicate mixture is not concentrated around a specific range, but presents a distribution covering all ranges of pore sizes. On comparing the pore size distribution of the non-exposed sample and the samples exposed for 12 months, one observes a progressive drop in the volume of pores with a diameter above 0.05 μm . Simultaneously, the volume of pores below 0.05 μm increases. This tendency is not followed by all

fractions of pores of the specimen tested after 6-month exposure; it appears that an increase in the pore volume of diameter ranging between 1.0-2.0 μm and 0.8-1.0 μm has taken place, while the volume of pores with diameter between 0.01-0.05 μm has decreased.

Table 3: Pore size distribution ($\text{cm}^3 \text{g}^{-1}$) of the silicate pastes and of the HOPC paste

diameter (μm)	P1			P2			P6		
	0m	6m	12m	0m	6m	12m	0m	6m	12m
<0.01	0.012	0.012	0.011	0.015	0.026	0.038	0.004	0.000	0.014
0.01-0.05	0.048	0.032	0.034	0.047	0.040	0.056	0.029	0.021	0.027
0.05-0.1	0.009	0.002	0.004	0.034	0.011	0.004	0.031	0.017	0.012
0.1-0.2	0.006	0.001	0.003	0.032	0.021	0.006	0.027	0.013	0.010
0.2-0.4	0.004	0.001	0.002	0.045	0.035	0.018	0.009	0.010	0.005
0.4-0.6	0.002	0.001	0.001	0.033	0.026	0.020	0.002	0.003	0.001
0.6-0.8	0.001	0.001	0.000	0.024	0.021	0.018	0.001	0.002	0.000
0.8-1.0	0.001	0.001	0.001	0.020	0.026	0.017	0.001	0.001	0.000
1.0-2.0	0.003	0.001	0.002	0.024	0.053	0.017	0.002	0.001	0.000
2.0-4.0	0.003	0.002	0.002	0.008	0.005	0.002	0.002	0.001	0.000
4.0-10.0	0.002	0.002	0.002	0.001	0.000	0.000	0.001	0.000	0.000
>10.0	0.003	0.002	0.004	0.011	0.006	0.007	0.002	0.001	0.002

Table 4: Pore size distribution ($\text{cm}^3 \text{g}^{-1}$) of the aluminate and aluminoferritic pastes

diameter (μm)	P3			P5		
	0m	6m	12m	0m	6m	12m
<0.01	0.004	0.001	0.007	0.011	0.008	0.007
0.01-0.05	0.008	0.010	0.009	0.022	0.019	0.016
0.05-0.1	0.000	0.000	0.001	0.007	0.014	0.015
0.1-0.2	0.000	0.001	0.001	0.009	0.001	0.003
0.2-0.4	0.000	0.002	0.002	0.007	0.001	0.004
0.4-0.6	0.002	0.004	0.002	0.003	0.004	0.003
0.6-0.8	0.002	0.004	0.002	0.001	0.006	0.002
0.8-1.0	0.004	0.004	0.005	0.006	0.002	0.002
1.0-2.0	0.022	0.017	0.018	0.000	0.000	0.003
2.0-4.0	0.068	0.033	0.077	0.000	0.000	0.001
4.0-10.0	0.129	0.134	0.033	0.000	0.000	0.000
>10.0	0.002	0.009	0.005	0.002	0.002	0.004

Concerning paste P3, the MIP results showed a high total porosity, of the same order of magnitude as paste P2. However, the pore size distribution is very different. A concentration of pores in the 4.0-10.0 μm diameter range was measured, which remained practically unaltered after 6-month exposure. During this period, the pores ranging between 2.0-4.0 μm seem to have been more affected and a significant drop has taken place. In the next 6 months of exposure, a sharp decrease in the pores between 4.0-10.0 μm was measured. The increase in the volume of pores in the 2.0-4.0 μm range and the simultaneous decrease of that in the 4.0-10.0 μm range highlight the progressive refinement of the pore structure. The other pore categories remain practically constant in volume with time. The pore size distribution, together with the aspect of the specimen (which seemed fragile and had a smooth, easily scratched

surface) suggests the hypothesis that the specimen had failed during testing. Whatever the case, it seems that a certain refinement has taken place.

The total porosity of paste P5 exhibited the lowest values of all tested pastes. It also seems that no important change in the total porosity volume took place. A drop in the latter was measured after 6 months of exposure, but the average results after 12 months indicated no significant change. In the case of P5, pores were concentrated in the range of diameters below 0.6 μm . A comparison between the non-exposed and the 12-month exposed specimen show that a progressive refinement in the pore ranges of 0.2-0.4 μm , 0.1-0.2 μm , 0.01-0.05 μm and lower than 0.01 μm had taken place. The refinement of the first two pore categories could explain the increased volume of pores around 0.05-0.01 μm . On the other hand, the pore volume occupied by pores bigger than 0.6 μm seems also to increase with time.

Table 5: Soluble sulphite and sulphate concentrations ($\mu\text{g g}^{-1}$) measured in the pastes non-exposed (0 m) and exposed for 6 and 12 months at 0.3 ppm SO_2

Sample	0 m	0 m	6 m	6 m	12 m	12 m	12 m
	SO_3^-	SO_4^-	SO_3^-	SO_4^-	SO_3^-	SO_4^-	$\text{SO}_3^-/\text{SO}_4^-$
P1	-	32	83	28	143	208	0.7
P2	-	44	70	66	171	196	0.9
P3	-	9	31	32	122	628	0.2
P5	-	3	11	41	84	332	0.2
P6	-	3063	47	2387	183	3046	0.06

The total porosity of the non-exposed paste P6 is $0.111 \text{ cm}^3 \text{ g}^{-1}$. A drop in the total porosity is also observed within the first 6 months of exposure, after which porosity seems to stabilise. The pores are distributed mainly in the range below 0.4-0.6 μm . In this area, a decrease in the pore volume of all pore categories is observed among the non-exposed and the 12 months exposed specimens, with the exception of pores with a diameter smaller than 0.01 μm . The volume of the latter increases sharply with time. Some porosity seems to remain in the higher pore ranges (over 0.6 μm).

Specific surface BET increases in all samples, except in P1, with exposure time. P2 shows the most important changes.

3.2 Chemical analyses

The samples, non-exposed as well as exposed, were analysed by means of ion chromatography (Dionex 4500i) to quantify the SO_3^- and SO_4^- water soluble contents, using a methodology specifically set-up (Gobbi et al., 1998). The data are reported in Table 5. The SO_4^- values at 6 and 12 months are reported after the subtraction of the SO_4^- amount of the non-exposed samples; the $\text{SO}_3^-/\text{SO}_4^-$ ratio is also shown.

In all samples a significant increase in the SO_4^- concentration can be observed, along with considerable quantities of sulphite. Furthermore the single pastes show a

higher relative reactivity in the period between 6-12 months, while the HOPC sample shows the highest reactivity in the first 6-month exposure period.

The evaluation of the SO_3^- content and trend permits the identification of the sulphation mechanism. The presence of sulphite indicates that sulphation occurs through two mechanisms: 1) the formation of sulphite as an intermediate product, followed by its oxidation into sulphate and 2) the direct formation of sulphate. The $\text{SO}_3^-/\text{SO}_4^-$ ratio with values >1 highlights the prevalence of a sulphation mechanism where sulphite formation is an intermediate stage preceding oxidation, while values <1 indicate the prevalence of a direct sulphation mechanism, generally enhanced by the presence of catalysts. The $\text{SO}_3^-/\text{SO}_4^-$ ratio reported in Table 5 indicates that in all pastes and particularly in the HOPC sample, with a ratio of one order of magnitude lower than in others pastes, mechanism 2 prevails, i.e. direct sulphation.

In order to determine the total amount of SO_2 reacting with the samples, the total (soluble and insoluble) sulphate was quantified after 12 months using a special technique set up in our laboratories (Gobbi et al., 1995). This methodology involves the complete solubilisation of the sample by treatment with strong cation resin with H cycle, which produces the oxidation into sulphate of any sulphite present; the quantification of total SO_4^- is then performed by means of ion chromatography.

On the basis of the results reported in Table 5, the total S-soluble species were evaluated as SO_4^- soluble by expressing SO_3^- as equivalent SO_4^- concentration, as follows:

$$\text{SO}_4^-(\text{sol}) = \text{SO}_4^- + [\text{SO}_3^- \text{ expressed as equivalent } \text{SO}_4^-] \quad (1)$$

The total SO_4^- data permit the calculation of the insoluble SO_4^- by the following relationship:

$$\text{SO}_4^-(\text{insol}) = \text{SO}_4^-(\text{tot}) - \text{SO}_4^-(\text{sol}) \quad (2)$$

In Fig. 1, total, soluble and insoluble SO_4^- is reported for the 5 exposed pastes after the subtraction of the SO_4^- found in the non-exposed paste.

The results indicate that sulphation is more intense for HOPC (sample P6) than for its single components. In fact, P6 sample shows the highest value of total SO_4^- . The soluble SO_4^- value is also highest in P6, while P3 presents the highest value of insoluble SO_4^- .

3.3 Mineralogical and microstructural characterisation

The main effect observed in the exposed P1 and P2 samples is their partial carbonation: calcite and calcite plus vaterite were respectively identified in P1 and P2 by XRD and FTIR. A CaCO_3 concentration of $\approx 25\%$ for P1 and $\approx 59\%$ for P2 was measured by TGA. SEM-EDX analyses show that the morphology of the exposed samples changes slightly when compared to non-exposed samples and that some sulphur was also found in the 12-month samples.

The increase in volume generated by the portlandite carbonation explains not only the drop in total porosity, but also the decrease in pores of the $0.01\text{--}0.05\text{ }\mu\text{m}$ pore fraction. It also justifies the BET specific surface reduction (Table 2). The sulphation of both samples is low (Fig. 1) and the carbonation process prevails due to the low product of solubility of CaCO_3 in relation to that of calcium sulphite or sulphate.

The FTIR spectra of sample P2 (Fig. 2) show a displacement toward higher frequencies of the band due to stretching vibrations of the Si-O bonds of the C-S-H gel.

This displacement is more intense when the exposure time increases, and it might be due to the decomposition of the C-S-H gel because of a carbonation process. An intense decrease in the combined water is observed by TGA, confirming the partial destruction of the C-S-H gel observed by FTIR. The result is that CaCO_3 formation reduces the total porosity and also the amount of large pores. Simultaneously, C-S-H gel decomposition releases a very fine SiO_2 gel, which increases the BET specific surface and the amount of the smallest pores.

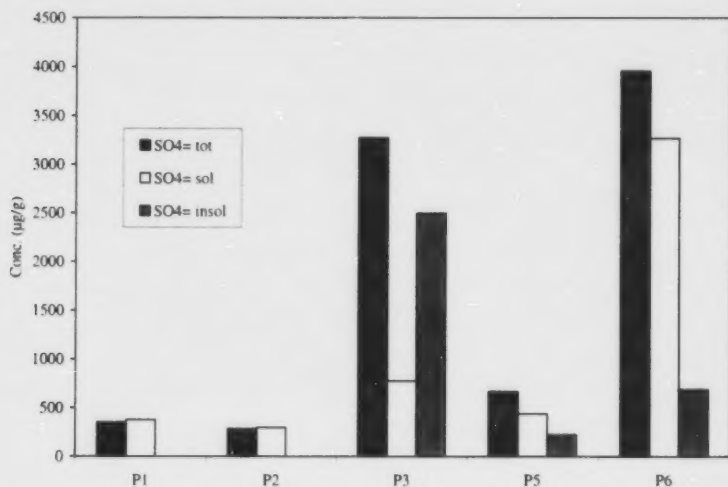


Fig. 1: Total, soluble and insoluble SO_4^{2-} formed on the pastes exposed 12 months at 0.3 ppm SO_2

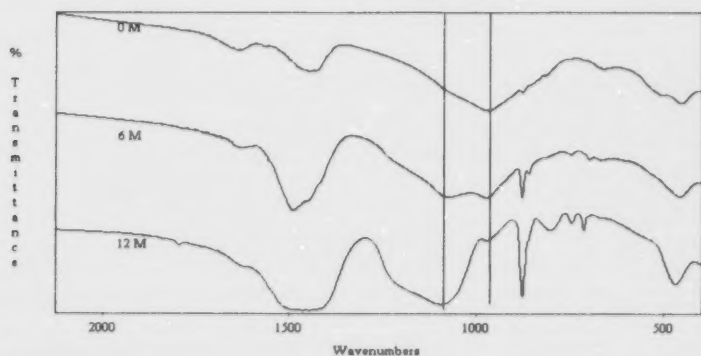


Fig. 2: FTIR spectrum of P2 paste at 0, 6 and 12 months of 0.3 ppm SO_2 exposure

XRD, TGA and FTIR results show no new compounds in samples P3 and P5 after exposure. However, important differences were detected by SEM-microstructural analysis in P3: at 6-month exposure and especially at 12 months, the characteristic morphology of the cubic hydrate (C_3AH_6) disappeared. P3 was also studied by TEM: microanalysis and electron-diffraction were applied. The sample turned out to be basically formed of two types of compounds: the most abundant is a calcium aluminate hydrate (C_3AH_6), whose crystals present no defined morphology; the other is a calcium sulphoaluminate hydrate. The morphology and analyses of these crystals (needle-shaped crystals) indicate that some ettringite has formed (Fig. 3). The size of ettringite crystals in P3 after 12 months of exposure is very small (~ 10 – 15 nm) and the amount low ($\sim 1\%$ wt., considering that all the insoluble sulphates would be ettringite). These are the reasons for which ettringite could not be detected using traditional tools (XRD and SEM).

No important changes in P5 morphology (after 6 or 12 months of exposure) were observed by SEM-microstructural analysis. According to SEM-EDX and TEM-EDX results, traces of sulphur at the sample surface were found, but no ettringite or mono-sulphoaluminate were present. This sample is the least porous and least reactive with SO_2 (Fig. 1). After 12 months of exposure, the maximum amount of gypsum and ettringite that could form (assuming all soluble and insoluble SO_4^{2-} is in the form of gypsum and ettringite, respectively) is: gypsum $\sim 0.07\%$ wt. and ettringite $\sim 0.1\%$ wt. The low reactivity justifies the absence of microstructural changes in P5 after the exposure tests. No carbonation process was observed during exposure tests, while sulphation processes of different intensity occur in P3 and P5.

P6 contained C-S-H gel, portlandite and ettringite at the beginning of exposure tests. During the test, P6 underwent both carbonation and sulphation processes; calcite and vaterite were identified by XRD and FTIR, while calcium mono-sulphoaluminate hydrate salt was also identified by XRD and TEM-EDX. Low amounts of Friedel's salt was seen by TEM. TGA data reveal that the 12-month exposed samples contained a higher percentage of $CaCO_3$ and a lower % of portlandite, compared with the non-exposed sample. Through FTIR studies, a progressive modification of bands due to the decomposition of the C-S-H gel was observed as a function of time. This modification might be due to the decomposition of the C-S-H gel because of carbonation and sulphation effects. Important changes in the morphology of P6 were observed by SEM. The samples at 6 and 12 months of SO_2 exposure presented an external layer. Some particles very rich in sulphur (which could be gypsum or some other amorphous calcium sulphates) were observed. The carbonation process justifies the porosity and BET specific surface evolution.

P6 is the most reactive sample with SO_2 (Fig. 1), although it is neither the most porous, nor the one with the highest specific surface. This behavior can be justified by the simultaneous precipitation of mono-sulphoaluminate and calcite. It appears to subtract Ca^{++} , $Al/Fe(OH)_4^-$, CO_3^{2-} ions to the solution, filling the porous system. On the one hand, it involves new dissolution and a higher production of SO_3^{2-} and SO_4^{2-} in the sample, while, on the other, it also involves new dissolution of material in order to generate Ca^{++} and $Al/Fe(OH)_4^-$ ions for re-establishing the equilibrium.

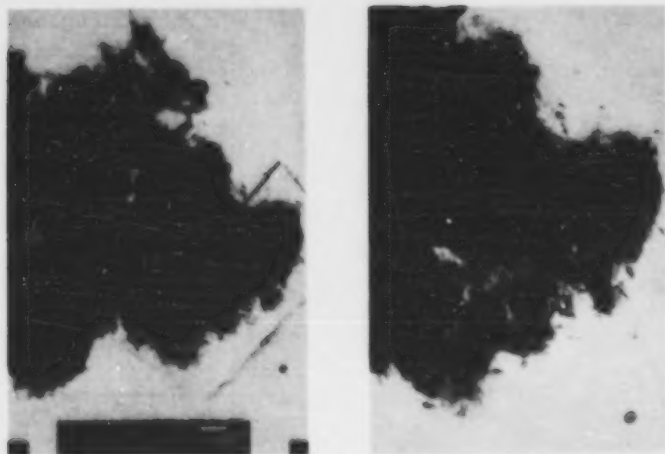


Fig. 3: TEM micrographs of P3 sample showing small crystals of ettringite

4. Conclusion

The results obtained from the SO_2 exposure tests on synthetic hydrated Portland cement (HOPC) and its single components showed that:

- i) The total porosity markedly decreases in all samples ($\approx 31\%$), with the exception of P5 (1%), due to its low water/solid ratio. Moreover, a pore refinement is observed in all the samples, becoming increasingly marked with the increase of exposure time. The decrease in the total porosity as well as the pore refinement are both caused by the deposition of substances in the pores due to reprecipitation or formation of new species through chemical reactions.
- ii) P6 and P3 samples show the highest reactivity with SO_2 , with a concentration of SO_4^{2-} (tot) of 3957 and 3270 $\mu\text{g/g}$, respectively, while P1 and P2 have the lowest reactivity (352 and 284 $\mu\text{g/g}$, respectively). Reactivity to SO_2 in environments with normal amounts of CO_2 is much more influenced by the chemical composition of the sample than by microstructure (porosity and specific surface). The presence of CO_2 does not enhance the reaction between calcium silicate hydrates and SO_2 because of the precipitation of CaCO_3 . This insoluble salt captures Ca^{2+} ions from the solution and consequently does not allow calcium sulphites and/or calcium sulphates to precipitate. When aluminates are present in the system, calcium sulphoaluminate is formed, which captures the SO_4^{2-} from the solution and favours new dissolution of SO_2 . The sulphur species formed on the samples during the SO_2 exposure have been found to be partially soluble and partially insoluble.

- iii) In all of the exposed samples, the soluble S-species were found as sulphites and sulphates. The amount of SO_3^- and SO_4^- is particularly high in P6 and P3. The presence of sulphite and the $\text{SO}_3^-/\text{SO}_4^-$ ratio (always <1) indicate that two sulphation mechanisms occur: 1) formation of sulphite as intermediate product followed by its oxidation into sulphate, and 2) the direct formation of sulphate. The second mechanism prevails, particularly in sample P6. Furthermore, all the samples show a high SO_3^- and SO_4^- trend in the period between 6 and 12 months; P6 shows the highest increase in the formation of soluble species during the first 6-month exposure.
- iv) In all of the exposed samples, the insoluble S-species were quantified as SO_4^- (insol), which was absent in samples P1 and P2, has the highest concentration in P3 (2493 $\mu\text{g/g}$) and is relatively low in P6 (601 $\mu\text{g/g}$). On P3 the insoluble S-species were identified as ettringite by TEM. The higher concentration of ettringite in P3 is explained by its chemical composition: P3 contains, in fact, the highest concentration of Al (i.e. 20%). The small amount of ettringite found in P6, despite the presence of a high SO_4^- (sol) amount (i.e. 3266 $\mu\text{g/g}$), may be due to the mass action law: P6 already has a large quantity of ettringite in the non-exposed P6 (i.e. 26985 $\mu\text{g/g}$), because of the gypsum (6%) present in the original components.
- v) The C-S-H decomposition, evidenced by FTIR analyses, highlights a competition between the carbonation and sulphation processes. In P1 and P2, carbonation prevails, with a concentration of CaCO_3 of 25 and 59%, respectively. In P3, P5 and P6 sulphation prevails, although in the latter a certain amount of CaCO_3 was also found.
- vi) These results, showing that sulphation reaction occurs on Portland cement components due to the deposition of atmospheric SO_2 , are particularly important as cement mortars have been used in 19th and 20th century monuments as well as in restoration works.

5 Acknowledgements

This study was supported by the EC, within the programme *Environment and Climate*, Project "Environmental Deterioration of Ancient and Modern Hydraulic Mortars - EDAMM" (Ct. n. ENV4-CT95-0096).

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25 EFFECT OF REPEATED LOAD ON MICRO STRUCTURE AND CARBONATION OF CONCRETE AND MORTAR

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Abstract

It is apprehended that durability of concrete and mortar subjected to repeated loads, even though they were in low level, may be lowered durability of them by fatigue failure in their micro structures. The effects of fatigue by flexure on micro structures and carbonation of them were studied in this paper. The three measurements such as dynamic modulus, micro pore volume and carbonation were carried out for the specimens fatigued at several levels of stresses. It was found out that the pore volumes of them increased, though dynamic moduli of them were little affected by the repeated loads. The deeper carbonation was also observed in the severely fatigued specimens and it seems to be closely related to change of their micro structures. To discuss quantitatively the effect of fatigue, a scale to estimate it is needed. Then, we proposed the following fatigue scale (D.F.) which indicates the degree of fatigue; $D.F. = \sigma / f + a (\log N)^b$, where σ ; flexural stress applied, f ; flexural strength, N ; number of repetition, and a, b ; constants. The micro pore volume increased and the depth of carbonation became deep with rise of value of the scale, which means fatigue progressed and thereby carbonation was accelerated. It is concluded that concrete and mortar subjected to repeated loads, even if they are at low levels, are changed in their micro structure by fatigue and thereby durability of them might be lowered.

Keywords: Carbonation, concrete, dynamic modulus, fatigue, flexure test, mortar, micro structure, repeated load

1 Introduction

Concrete used for structures, is subjected to numerous number of repeated loads during their service life. The performance of concrete to repeated load is commonly determined by fatigue tests, and most of them have been carried out mainly for determining the limitation of fatigue rupture of concrete, because structures become seriously dangerous according as mechanical property of concrete closes on its fatigue limitation.

Since members of a structure are designed to keep the stresses occurred in concrete beneath lower level than those of failure limitation, it is therefore quite seldom for concrete in actual structures to be subjected to such a higher level repeated load that may result in fatigue rupture. However, repeated loads give concrete a gradual damage even though the level of stress is low, and it would lead to reduction of durability of concrete. Because, the micro structure of concrete might be changed and thereby permeability of concrete to carbon dioxide gas would be presumed to increase.

Most previous work has focused on fatigue failure of concrete. As for fatigue itself, the effect of fatigue on the relation between pore structure and mechanical properties of concrete was investigated recently. However, there is still few knowledge on effect of fatigue, in particular to durability.

In this paper, the effect of low level fatigue on durability of concrete and mortar, in particular, on change of micro structures and carbonation by repeated load were investigated.

2 Test specimen

Prisms of 10cm×10cm in cross section and 40cm in length were used for all fatigue tests. The mix proportions of concrete and mortar specimens were shown in Table 1. They were kept in water of 20±2°C for three months to expect sufficient cure, and then dried in the room at 20±2°C and 60%R.H. for more than one month.

Table 1: Mix designs of concrete and mortar specimen

Materials	Water/ Cement ratio, %	Sand/ Aggregate ratio, %	Unit weight, kg/m ³				Dosage of AE& WR agent C×%	Slump (Flow), cm	Air content, %
			Water	Cement	Sand	Gravel			
Concrete	60	44	180	300	777	1002	0.10	18.0	5.8
Mortar	60	-	250	417	1554	-	-	(23.0)	4.5

3 Cyclic loading test for determine limitation of fatigue failure

3.1 Measurement of flexural strength

The data of flexural strengths of concrete and mortar are needed to decide the level of load in cyclic loading tests, and therefore flexural tests were carried out in conformity of the JIS Standard A 1106. The mean values of averaged flexural strength of five specimens were 6.67N/mm^2 for concrete and 5.06N/mm^2 for mortar respectively.

3.2 Cyclic loading tests for rupture

Cyclic loading tests were carried out as shown in Photo.1, to obtain a fatigue rupture curve which is used for deciding a level of stress and a number of repetition for further cyclic loading tests for fatigue. The upper ratios in the stress level were 0.8, 0.7, and 0.6 of the flexural strength of each material. The specimens were repeatedly loaded between the upper levels to zero at 5Hz, and the numbers of cycles at rupture were counted.

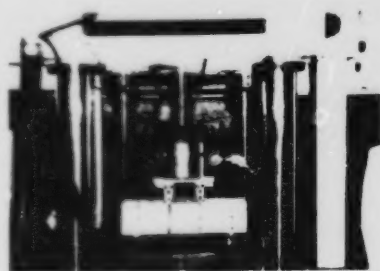


Photo. 1: Cyclic loading test

The relations between the levels of stress and the numbers of cycles to rupture were shown in Fig.1 for the concrete and mortar specimens. The numbers of cycles to rupture reasonably decrease as the levels of stress rise. It can be seen that the results have a smooth relation between the levels of stress and the logarithmic values of number of cycles for the both materials, and then their relations can be expressed by the following empirical equation,

$$\sigma/f = 1 - a (\log N)^b \quad (1)$$

where, σ : applied flexural stress, N/mm^2
 f : flexural strength, N/mm^2
 N : number of repetition to rupture
 a, b : constants.

The two constants were determined from the fatigue test results of concrete and mortar, and the equations expressing the relation between a level of stress and a number of cycles to rupture were obtained as shown respectively in Table 2.

3.3 Cyclic loading tests for fatigue

Considering the former relations, levels of stress and numbers of repetition for cyclic tests for fatigue, in which fatigue rupture is not expected, were decided as shown in Table 3.

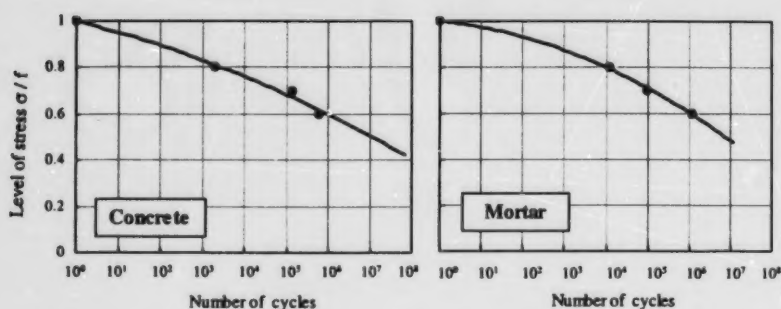


Fig. 1: Relation between level of stress and number of repetition to rupture

Table 2: Equations expressing fatigue rupture

Materials	Equations
Concrete ($f: 6.67 \text{ N/mm}^2$)	$\sigma/f = 1 - 0.0431(\log N)^{1.24}$
Mortar ($f: 5.06 \text{ N/mm}^2$)	$\sigma/f = 1 - 0.0196(\log N)^{1.68}$

Table 3: Stress level and number of repetition of cyclic test for fatigue

Concrete					Mortar				
Stress level σ/f	Number of repetition ($\times 10^6$)				Stress level σ/f	Number of repetition ($\times 10^6$)			
	0.1	0.4	1	2		1	2	5	10
0.3			✓	✓	0.3	✓			
0.4			✓	✓	0.4	✓			
0.5			✓		0.5	✓	✓	✓	✓
0.6		✓			0.6	✓			
0.7	✓								

✓: A test was carried out.

4 Results of cyclic loading test for fatigue

4.1 Change of dynamic modulus by fatigue

Dynamic moduli of 4 concrete specimens were measured before and after the cyclic loading tests to make clear the effect of fatigue on mechanical property of them. The measurement was carried out by a primary deflection method described in JIS A 1127 and the results thus obtained were shown in Table 4. It can be seen from the table that the dynamic moduli of them decreased by fatigue. However, the difference of them between before and after the fatigue operation was slight.

4.2 Change of micro structure by fatigue

Micro structures of concrete and mortar specimens were also studied before and after the fatigue operation to make clear the effect of fatigue. Pore size and distributions of them were measured in the size from 6nm to 180 μ m in diameter through the Micromeritics Poresizer 9310.

The procedure is as follows. A specimen is first divided into two pieces at the center of it. Five small blocks are then cut out from five positions of the divided piece as shown in Fig.2, (the other half of piece is used for an accelerated carbonation test). The five blocks have been subjected to different kinds and different levels of stress during a fatigue operation, such as maximum compressive stress, maximum tensile stress, composite stress of compressive and shear, composite stress of tensile and shear, and less stress because of out of the loading points. As for concrete specimens, they contain coarse aggregates in them, and therefore, they were crushed and small blocks of only mortar portion of 2.5mm-5mm in size were used to the measurement.

The pore distributions at maximum tensile stressed and less stressed positions were shown in Fig.3. Much pore volumes of the samples from the area of being stressed were observed, compared to that of less stressed area, and the difference between them is more distinguished in smaller pore size regions.

For discussing the difference of pore distributions among the positions in a specimen more clearly, the results were reformed to bar charts as shown in Fig.4, in which pore sizes are divided into the four zones, i.e., a very fine capillary pore zone(6-10nm), a smaller capillary zone(10-100nm), a medium capillary zone(0.1-5.68 μ m) and a larger capillary and entrained air zone(5.68-180 μ m).

Table 4: Dynamic modulus of elasticity of concrete before and after fatigue test

Stress level σ/f	Number of repetition ($\times 10^4$)	Dynamic modulus of elasticity, $\times 10^4 \text{N/mm}^2$		Reduction percentage of dynamic modulus, %
		Before fatigue	After fatigue	
0.3	2	3.74	3.73	0.3
0.4	2	3.70	3.70	0
0.5	0.4	3.71	3.70	0.3
0.6	0.1	3.74	3.71	0.8

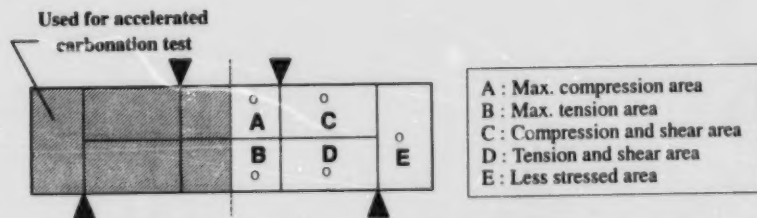


Fig. 2: Sampling position (shown by circles) for pore distribution measurement

Much pore volumes at the positions subjected to higher levels of stress were observed, compared to that of less stressed position for both concrete and mortar specimens. The pore volumes of pore sizes of 10nm-100nm and 0.1 μ m(100nm)-5.68 μ m were particularly increased.

4.3 Change of carbonation rate by fatigue

There is fear that fatigue may lower durability of concrete and mortar, in particular the resistance to carbonation. Then, an accelerated carbonation test was carried out to study the effect of fatigue on carbonation. The remained half of the specimen as mentioned above was used for the test. The four sides of a specimen except the upper and the bottom surfaces where force had been directly applied, were sealed by epoxy resin coatings to prevent from ingress of carbon dioxide gas during the carbonation test.

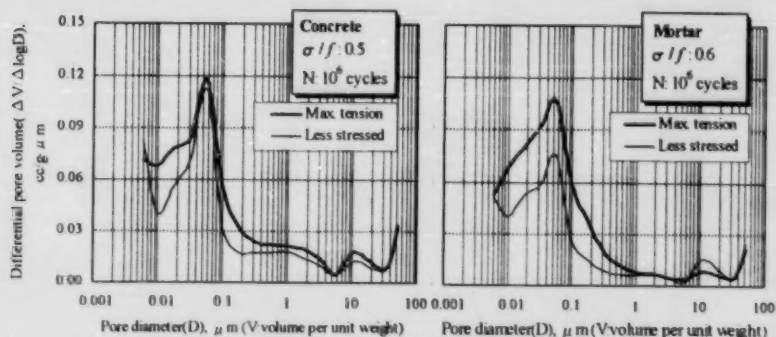


Fig. 3: Pore size distribution of tensile stressed area and less stressed area

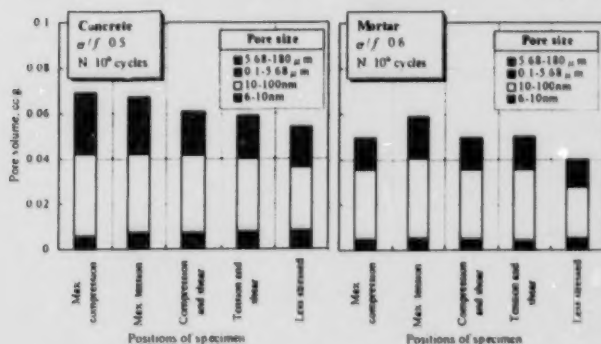


Fig. 4: Volume of four pore size zones at various stressed positions

After being exposed in a chamber filled with carbon dioxide gas of 20% in CO_2 concentration at 20°C and 60%R.H. for 2 months, the specimen was split into two pieces along the longitudinal direction by a compressive test machine. Phenolphthalein liquid reagent was sprayed over the split surface of the specimen, and the carbonated depth was measured at distance of 5mm along the longitudinal direction of the specimen. As for concrete, carbonated depth was measured in the area of mortar avoiding coarse aggregates, because a coarse aggregate does not inherently carbonate. The results were expressed by averaged depth of the measured area as shown in Fig.5. It shows that the carbonation depth of the areas subjected to higher levels of stress become deeper than those of less stressed for both materials.

5 Method for evaluating fatigue

As mentioned in the previous section, fatigue rupture is mainly affected by a level of stress and a number of repetition, and the relation of them was roughly expressed by the equation (1). When the right side of the equation transposes to the left side, it becomes as follows;

$$\sigma/f + a (\log N)^b = 1 \quad (2)$$

This equation means that a specimen does not rupture until the value of the left side of the equation, $\sigma/f + a (\log N)^b$, reaches the value of 1. Therefore, the value of the left side of the equation is considered to use as a scale that indicates progress of fatigue of concrete and mortar specimens. We define it as the degree of fatigue, D.F., it is able to be expressed by the following equation,

$$\text{D.F.} = \sigma/f + a (\log N)^b \quad (3)$$

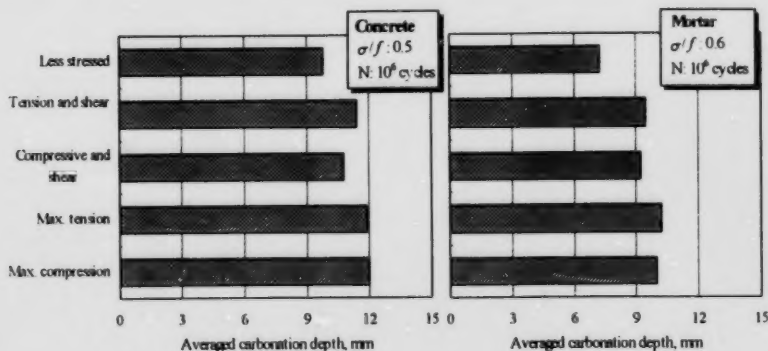


Fig. 5: Averaged carbonated depth of positions subjected to different stress

A level of stress and a number of repetition, both of which are important parameters in a fatigue test, are able to be unified by the equation (3). The degrees of fatigue were calculated by the equation (3) for the fatigued specimens, and they are shown by numeral values in Fig.6.

6 Discussion

6.1 Effect of fatigue on dynamic modulus of concrete

Change of dynamic modulus, which is expressed as the ratio of the value after fatigue test to original one, is shown for concrete specimens in Fig.7. It seems that fatigue little affect dynamic modulus of concrete.

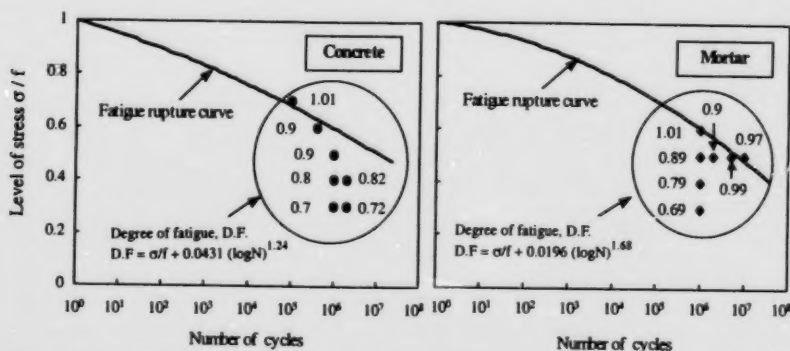


Fig. 6: Degree of fatigue (D.F.) of specimen subjected to various levels of fatigue

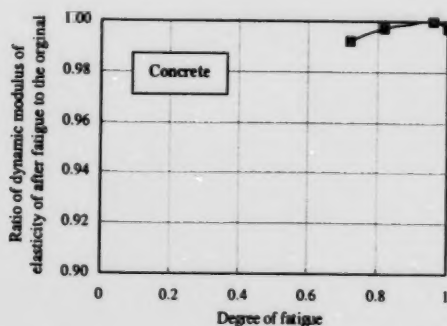


Fig. 7: Relation between dynamic modulus of elasticity and degree of fatigue

6.2 Effect of fatigue on micro structure of concrete and mortar

The relation between pore volumes, which are divided into four zones in pore size, and degree of fatigue was studied. The changes of pore volumes in maximum compressive zone and maximum tensile one were shown in Fig.8 for concrete and Fig.9 for mortar respectively. It can be seen a tendency that total pore volume increases with the degree of fatigue. As for the pore sizes that changed by fatigue, the volumes of capillary pores from 10 to 100nm and 0.1 μ m to 5.68 μ m were increased. The effect of fatigue on micro structures is able to understand as an increase of micro pores.

6.3 Effect of fatigue on carbonation

The relation between carbonation depth of the two materials and degree of fatigue was also investigated. Fig.10 shows their relations in the positions of maximum compressive, maximum tensile and less stressed.

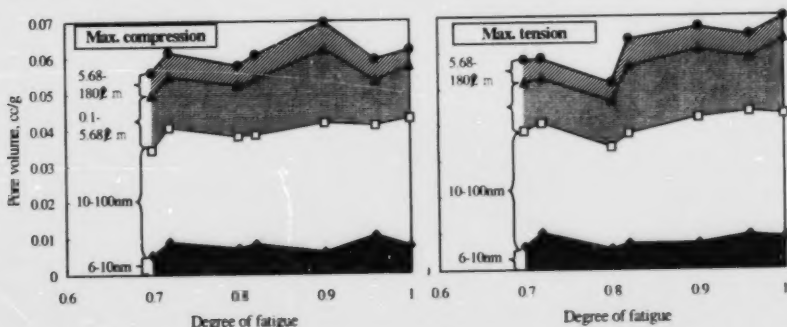


Fig. 8: Relation between pore structure of concrete and degree of fatigue

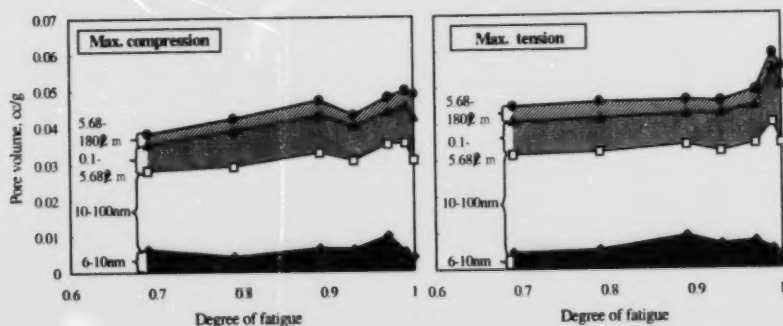


Fig. 9: Relation between pore structure of mortar and degree of fatigue

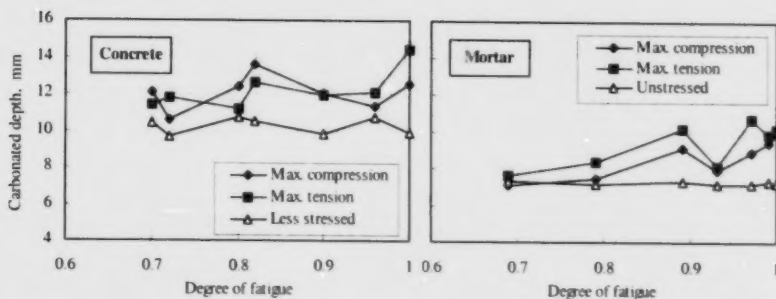


Fig. 10: Relation between carbonation depth and degree of fatigue

The carbonation depth at less stressed hardly changed, however, those at the positions severely stressed such as maximum compression and tension areas, become deep with increase of the degree of fatigue. It is said that carbonation of concrete depends on the pore volume of it, and the deeper carbonation in stressed positions in the tests is considered to be due to an increase of capillary pores of 10-100nm and 0.1-5.68 μ m.

7 Conclusion

The Effect of repeated loads on pore structure and carbonation of concrete and mortar was studied, and the following conclusions were obtained.

- (1) Concrete and mortar subjected to repeat loads, even if they are at low levels, are changed in their micro structures by fatigue and thereby become more sensitive to carbonation.
- (2) A scale of expressing fatigue, which consists of a level of stress and a number of repetition, was proposed. The usefulness of the scale for evaluating degree of fatigue for concrete and mortar was shown through comparing the change of pores and the carbonation depth of them by fatigue.

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PART ONE

**DURABILITY OF CONCRETE
MATERIALS AND
STRUCTURES**

CORROSION OF CONCRETE



26 IN-SITU ASSESSMENT OF CORROSION INDUCED DAMAGE OF THE DICKSON BRIDGE DECK

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Abstract

The assessment of corrosion induced damage in the Dickson bridge deck was part of a collaborative research program between McGill University and The Queen's University of Belfast, along with some industrial partners. This program was aimed at identifying the causes of deterioration, and investigate suitable in-situ test techniques for predicting corrosion. This paper describes the results of the detailed tests are presented summarily. The chloride content at the level of steel was found to be very high and hence the main cause of steel corrosion. The analysis results showed that the concrete strength is not a suitable means of predicting deterioration, however permeability, resistivity or diffusivity of the concrete and its exposure to salt are more likely to provide a useful guide to chloride induced corrosion.

Keywords: In-situ testing, steel corrosion, assessment, diffusivity, permeability.

1 Introduction

This paper reports on a corrosion survey of a 39-year old reinforced concrete bridge subjected to a severe environment. Construction of the Dickson bridge was completed in 1959 at a cost of under a million dollars in a heavily industrial zone as a grade separation over the CN rail tracks in the east end of Montreal. The bridge consists of three sections, with the north and south sections consisting of reinforced concrete deck on concrete beams and girders on concrete column piers and the central section consisting of reinforced concrete deck on steel girders on steel column piers. The City of Montreal decommissioned the bridge in 1994 due to the extensive

deterioration of the bridge deck, and it was made available in late 1996 for investigation of the causes of deterioration and for developing a detailed data bank of the various parameters influencing corrosion of the reinforcing steel.

2 Objectives and methodology of investigation

The primary objectives of the testing programme were as follows: (a) to assess the corrosion induced damage in the concrete deck in the northern part of the bridge, (b) to identify the causes of the observed deterioration, and (c) to investigate suitable test techniques for predicting corrosion.

A range of test methods was used. These are methods that either are related to the mechanisms causing the corrosion or properties indicative of the probability of corrosion. The susceptibility to deterioration is governed by the material characteristics and the exposure parameters, while the damage is measured using the available techniques. Hence, the first part of the investigation emphasized methods for assessing the probable damage to the concrete structure. A thorough visual inspection included detection of delamination by tapping the surface with a hammer, and the location of the reinforcement and the depth of the concrete cover using a covermeter. The extent and rate of corrosion of the reinforcing steel were measured using the half-cell potential and linear polarization, respectively. The chloride content at the level of the reinforcement was determined using the acid-soluble chloride technique while the depth of carbonation was measured using the phenolphthalein test. The second half of the investigation evaluated the properties of concrete, such as its compressive strength (from drilled cores) and durability characteristics, namely the permeability, diffusivity and resistivity of the concrete. The permeability was measured using the Autoclam permeability system while the diffusivity was determined both from the chloride profiles and by using a recently developed in-situ chloride migration test (Basheer 1992; Basheer et al. 1995). The electrical resistivity was then evaluated using the Wenner four probe apparatus.

A summary of the field test results is presented along with the condition of the bridge as evaluated. Full details of this investigation are presented by Fazio (1999), Mc Cafferty (1998), and Andrews (1999).

3 Description of the condition of the bridge from field test results

The bridge deck was inspected for the varying levels of concrete deterioration related to the different degrees of corrosion of the steel reinforcement. Presence of rust stains, spalling of the concrete cover, exposure of the reinforcing steel, and loss of the steel cross-sectional area were noted at several locations. This was noticed mostly towards the crest of the bridge. The west side of the bridge deck showed more damage, especially on the underside of the bridge deck. The expansion joints situated at grid locations 37 m and 127m (Fig. 1) were found to be leaking and there were signs of run down from the bridge deck to the underside of the deck.

The basic descriptive statistics of all the experimental data are summarized in Table 1. The findings show a large variance in some of the parameters such as the apparent chloride diffusivity coefficient (D_a), and the coefficient of permeability (k_a) (Table 1, COV 1). Therefore, before undertaking any regression analysis of the various parameters involved, the distribution of the data of each variable was examined for signs of normality, outliers and skewness. The COV of this analysis is included in Table 1 (COV 2). Appropriate transformations were applied to the parameters when the COV was large in order to improve the agreement with normality (Table 1, COV 3).

Table 1: Results of experimental data

Phenomenon	Analysis 1				COV 1	COV 2	COV 3
	No. of Readings	Max Value	Min Value	Mean Value			
Half-cell survey (mV)	2484	-44	-750	-407	0.23	-	-
Open circuit potential (mV)	137	-123	-700	-444	0.33	-	-
Corrosion rate ($\mu\text{m}/\text{year}$)	137	130	4	52	0.55	0.48	0.16
Delamination	140	-	-	-	-	-	-
Concrete cover thickness (mm)	137	90	1	35	0.48	0.45	-
Percentage chloride at steel level	35	2.20%	0.07%	0.73%	0.72	0.60	-
Apparent diffusion coefficient (cm^2/s)	32	51.6×10^{-9}	1.4×10^{-9}	9.3×10^{-9}	1.02	0.62	0.40
In-situ migration coefficient (cm^2/s)	35	13.5×10^{-8}	1.6×10^{-8}	6.3×10^{-8}	0.48	0.46	0.30
Electrical resistivity ($\text{k}\Omega\text{cm}$)	137	27	1	6	0.72	0.43	0.35
Coefficient of permeability (m/s)	34	84.2×10^{-14}	1.8×10^{-14}	16.3×10^{-14}	1.18	0.76	0.37
Compressive strength, (MPa)	34	48	10	27	0.36	-	-

3.1 Corrosion potential

A contour map of the entire north portion of the bridge is shown in Fig. 1 in which the potential ranges (ASTM C876, 1991) are indicated with different shades. The potential survey was undertaken over a grid of one meter by one meter. Table 2 lists the number of readings in each potential range, and the percentage distribution of these readings. The potential measurements indicated active corrosion of embedded reinforcement in most of the deck. Fifty-one percent of the readings suggest that there is a 50% to 90% risk of corrosion, while 25% of the readings suggest that the risk of corrosion is more than 90%. The low values of potential are concentrated in the first 33 rows of the grid (grid lines 0 to 33 m). The half-cell potential gradients are quite pronounced (Fig. 1). The very steep half-cell potential gradients indicate extensive pitting corrosion at local anodes. This is common for structures contaminated by deicing salts. The potential become more negative as the moisture content of the concrete varies, but it's important to realize that it does not influence the potential contours (Vassie, 1991). The relative humidity (RH) was observed to be quite consistent throughout the bridge and high in value. The RH at depths 10 mm and 25 mm from the concrete top surface were found to be on the average 85% and 90%, respectively. The west side of the bridge has a larger number of readings in the range of -500 mV and algebraically smaller potential values. The results show that the west side is deteriorating more than the east side (Table 2).

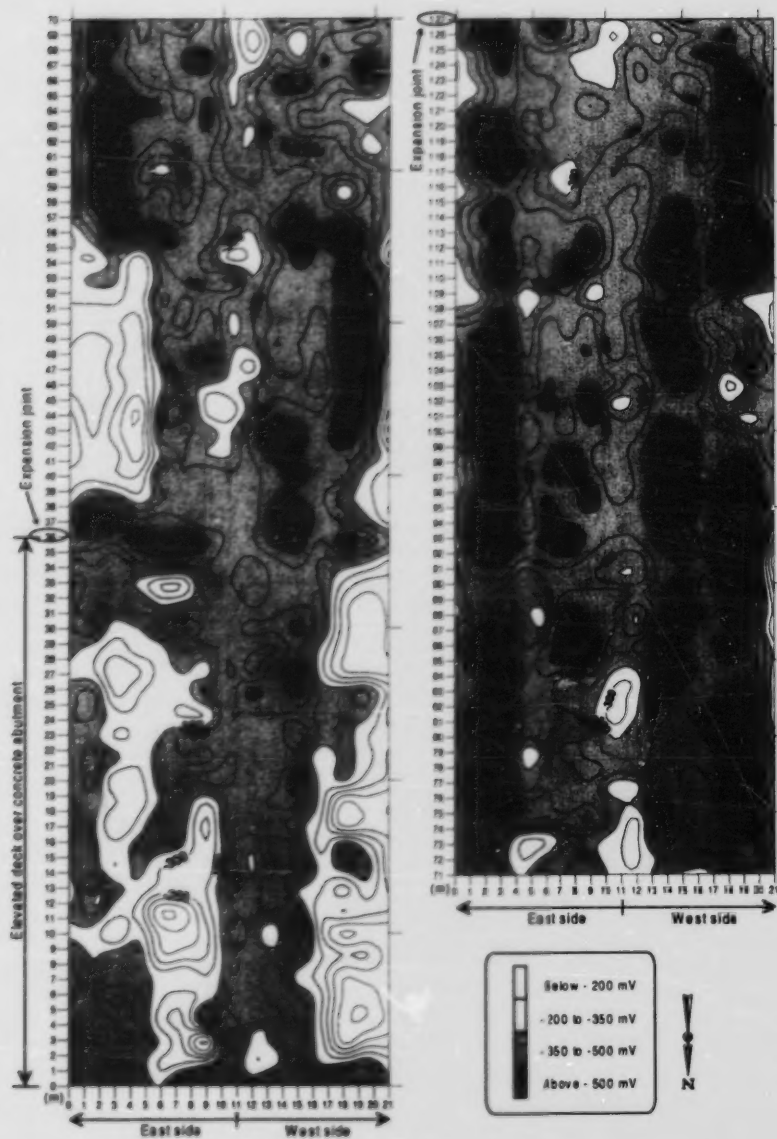


Fig. 1: Half-cell potential survey of the northern portion of the bridge deck

Table 2: Distribution of test results

Phenomenon	Criteria	Whole deck (%)	East side (%)	West side (%)
Half-cell (Open circuit potential)	Above -500 mV	25 (42)	19 (35)	34 (47)
	-350 to -500 mV	51 (31)	52 (31)	50 (32)
	-200 to -350 mV	22 (21)	27 (23)	14 (19)
	Below -200 mV	2 (6)	2 (11)	2 (1)
Corrosion rate	Below 1 µm/year	-	-	-
	1 to 10 µm/year	1	3	-
	10 to 45 µm/year	47	52	43
	45 to 100 µm/year	44	37	50
	100 to 1000 µm/year	7	8	7
Delaminated		53	34	71
Non-delaminated		47	46	29
Concrete cover thickness	Below 25 mm	29	37	22
	25 to 40 mm	30	26	33
	40 to 50 mm	26	18	32
	Above 50 mm	15	18	13
Chloride content at steel level	Below 0.2%	17	29	6
	0.2 to 0.4%	11	18	6
	0.4 to 0.6%	17	12	22
	Above 0.6%	54	41	67
Apparent diffusion coefficient	Below $0.3 \times 10^{-8} \text{ cm}^2/\text{s}$	19	25	13
	0.3 to $1.3 \times 10^{-8} \text{ cm}^2/\text{s}$	59	56	63
	1.3 to $2.3 \times 10^{-8} \text{ cm}^2/\text{s}$	16	19	13
	Above $2.3 \times 10^{-8} \text{ cm}^2/\text{s}$	2	0	13
In-situ chloride migration coefficient	Below $3.0 \times 10^{-4} \text{ cm}^2/\text{s}$	11	12	11
	3.0 to $6.0 \times 10^{-4} \text{ cm}^2/\text{s}$	37	53	22
	6.0 to $9.0 \times 10^{-4} \text{ cm}^2/\text{s}$	29	18	39
	Above $9.0 \times 10^{-4} \text{ cm}^2/\text{s}$	23	18	28
Electrical resistivity of concrete	Below 5 kΩcm	52	52	51
	5 to 10 kΩcm	31	25	36
	10 to 20 kΩcm	15	18	13
	Above 20 kΩcm	2	4	0
Coefficient of permeability of concrete	Below $10.0 \times 10^{-14} \text{ m/s}$	56	50	61
	10.0 to $20.0 \times 10^{-14} \text{ m/s}$	21	25	17
	20.0 to $30.0 \times 10^{-14} \text{ m/s}$	9	13	6
	Above $30.0 \times 10^{-14} \text{ m/s}$	15	13	17
Concrete compressive strength	Below 20 MPa	29	31	28
	20 to 30 MPa	29	31	28
	30 to 40 MPa	29	31	28
	Above 40 MPa	12	6	17

The half-cell potential readings were used to select thirty-five sites, measuring approximately 1 m by 2 m, for further detailed analysis. These sites corresponded to positions of highest and lowest potentials and the steepest potential gradients. A contour map was not possible to be plotted for the open circuit potential (from linear polarization test) as the readings were taken at 137 discrete locations (4 readings per site) and not at equal intervals. Forty-two percent of the readings are smaller than -500 mV and they predominate the top portion of the grid from grid locations 70 m to 127 m (Fig. 2(a)). Again, the west side of the bridge deck displayed a greater percentage of readings in this range compared to the east side (Table 2), which is in agreement with the results of the half-cell potential survey.

3.2 Corrosion rate

The corrosion rate is considered to be a quantitative measure of the amount of steel turning into oxide thus causing a reduction in the rebar diameter. Using Faraday's equation, corrosion current density was converted into metal section loss, that is into reduction of the average rebar diameter. The corrosion rates are plotted in Fig. 2(b) and show 91% of the readings indicate a high risk of corrosion based on a broad criteria for corrosion developed from the field and laboratory investigations with the sensor controlled guard ring device (Millard 1993). Similar results are depicted on the east and west sides of the bridge. However, it was also observed that 52% of the readings lie between 10 and 45 $\mu\text{m}/\text{year}$, and 37% between 45 and 100 $\mu\text{m}/\text{year}$ for the east side, and only 43% and 50%, respectively, in the west side (Table 2). As a whole, the rebar corrosion rates for the west side are higher. The readings of the bottom half of the grid are lower than the values for the top half (Fig. 2(b)).

3.3 Delamination

The horizontal cracks were detected by a hollow sound when the surface was hit with a hammer. Figure 2(c) shows the delamination results of the 140 selected areas (4 readings per site) throughout the northern part of the bridge. The delamination was very extensive, with half of these areas being delaminated. These are mostly located from grid locations 70 m to 128 m. The non-delaminated areas are concentrated in the first 40 meters of the grid. Seventy-one percent of the west side areas are delaminated while only 34% of the selected areas in the east side sites are delaminated (Table 2).

3.4 Concrete cover to steel

The concrete cover thickness was determined using the covermeter at the same locations where the corrosion rate was measured. Table 2 indicates that 29% of the readings are smaller than 25 mm, which was the cover thickness specified in the original design. At many locations the cover was found to be inadequate. Extreme values as low as only 1 mm in thickness were observed near the bottom portion of the grid (Fig. 2(d)).

3.5 Chloride percentage at steel level

Acid soluble chloride contents were measured using a chloride ion specific ion electrode (Herald et al. 1992). The threshold acid-soluble chloride content value (percent by weight of cement) reported by the ACI Committee 222 (1985) is 0.2%. The high values found in the bridge deck are due to the ingress of chlorides from the use of deicing salts during the winter period. From the profiles, it can be shown that 83% of the data lies above the 0.2% threshold value. The higher chloride percentages are concentrated more on the west side of the bridge as shown in Fig. 2(e) and Table 2. Figure 2(e) shows that the low chloride values are mostly in the bottom portion of the grid.

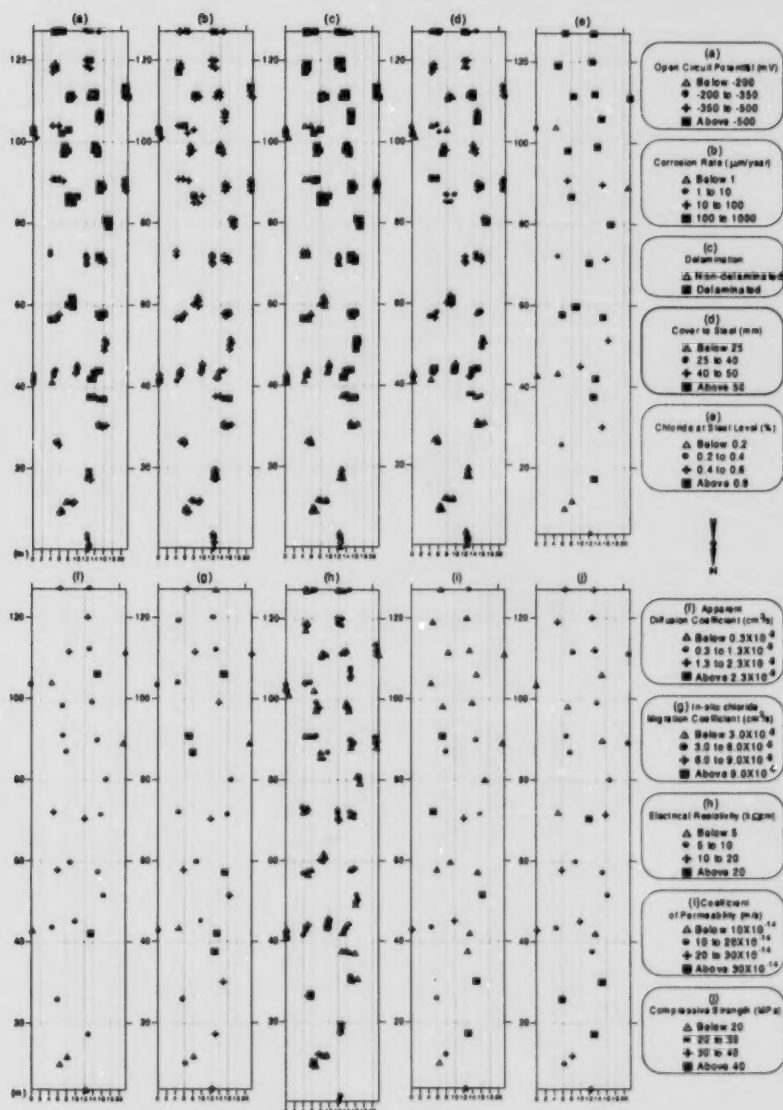


Fig. 2: Visual presentation of experimental data results

3.6 Diffusivity of concrete

3.6.1 Apparent chloride diffusion coefficient

The apparent chloride diffusion coefficient (D_a) values calculated from the chloride profiles have a COV of 1.20, indicating great variance in the results. The error function solution to Fick's second law of diffusion is commonly used to determine D_a from chloride profiles. Many researchers have investigated diffusivity and obtained widely varying results, as the D_a value is influenced by several factors such as chloride concentration, exposure duration, ambient temperature and degree of saturation of the concrete as well as the material properties, making it difficult to compare different published works. Regardless of the present state-of-the-art, the data can be compared on a relative basis within itself. The D_a values are plotted (Fig. 2(f)) using the ranges listed in Table 2. It is observed that the values of D_a lie mostly in the range 0.3 to 1.3×10^{-8} cm²/sec and it is distributed evenly between the west and the east side.

3.6.2 In-situ chloride migration coefficient

This variance in the in-situ chloride migration coefficient, D_{mig} , is lower than that for D_a , indicating that on site the former is a more appropriate parameter to measure the diffusivity of the concrete. This phenomenon is being investigated presently at Queen's University. However, compared to Andrews' (1999) results from tests at Queen's University, the values obtained for the bridge were much higher because of the exposure conditions, however, they can be analyzed on a relative basis (by comparison within the measured values). The D_{mig} values are plotted in Fig. 2(g) using the ranges used in Table 2. It is noted that a very high percentage of the results smaller than 3.0×10^{-8} cm²/sec exist on the east side, showing that the in-situ chloride migration coefficient values are higher in the west side of the bridge.

3.7 Electrical resistivity of concrete

Figure 2(h) illustrates the electrical measurements using the ranges described by Millard (1991). Fifty-two percent of the values lie in the very high probable rates of corrosion activity (Table 2). It is evenly distributed among the east and west sides of the bridge, and larger concentration values were observed near the top portion of the grid.

3.8 Coefficient of permeability

There is considerable variability in the coefficient of permeability results (k_w). A direct comparison with the data developed at Queen's University was not possible as the tests by Basheer (1992) and others used dry concrete. As the ambient relative humidity was quite high (average of 90%) and the concrete could not be dried to a constant moisture content, a steady state of flow during the permeability test was achieved by ponding water in the ring for 48 hours prior to testing. Montgomery and Adams (1985) undertook some work using ponded specimens, however, they used a pressure of 1.76 bars which is much larger than the value used in this investigation (0.5 bar). The k_w values obtained in this investigation were very small, which

indicated that the concrete is not permeable, however, it must be noted that the water pressure applied during test was only 0.5 bar. This pressure is lower than the surface tensional forces in capillary pores filled water hence no significant amount of water is able to penetrate through the saturated concrete. The permeability results should be considered with caution because the environmental conditions (RH) at the time of testing can significantly influence the results. The permeability values are plotted in Fig. 2(i) using the ranges listed in Table 2. It is noted that 56% of the values are under 10.0×10^{-14} m/s, and no particular trend is observed between the west and the east sides of the deck.

3.9 Compressive strength

The concrete compressive strength determined using 75 mm diameter cores varied from 10 to 48 MPa with a mean value of 27 MPa and a COV of 0.36 (Table 1). The specified concrete strength was 28 MPa. It is noted that 58% of the strength values were below 30 MPa (Table 2). The low values of strength are concentrated near the top portion of the grid and they are evenly distributed between the east and the west sides of the deck (Fig. 2(j)); the values near the bottom grid range between 40 and 60 MPa.

4 Discussion and conclusions

Overall, all of the methods that assess damage, such as potential, corrosion rate, delamination, concrete cover and chloride content indicate that the risk of corrosion in the top portion of the grid is high. Areas of high chloride content generally coincided with locations of significant reinforcement corrosion. Below the expansion joint (grid location 36 m), the damage is less. In addition, the damage is more pronounced in the west side of the deck.

The areas of high diffusivity, and low resistivity and strength, coincided with the locations of significant reinforcement corrosion. Thus, the early evaluation of the properties of the near-surface concrete, along with the knowledge of exposure to salt may provide useful indications of the likely future deterioration of the concrete. Although the various assessment methods were indicative of the magnitude of corrosion damage, they cannot detect the susceptibility to corrosion. Many concrete structures built in the 1960s and 1970s are now showing significant signs of deterioration. The Dickson bridge is a typical example. Measurement of the strength and workability is not sufficient to prevent the occurrence of these problems. It is evident, therefore, that some other property of concrete, which is more closely related to corrosion, needs to be monitored to effectively overcome this problem. Permeability, diffusivity and resistivity are such properties which have been found by many researchers and confirmed in this investigation to give much more useful information on the likely durability of concrete. Therefore methods to predict durability should be included in inspections of the bridge stock. The reliability of the test methods used in this investigation is discussed by Fazio (1999). No single test method will detect or predict the damage, however, when a combination of test

methods is used, it is unlikely that any centre of deterioration will get missed. A multivariable statistical analysis is the most appropriate way of quantifying and correlating the results of the various tests. Both methods (measuring damage, and measuring material properties and exposure parameters) have been used to predict the corrosion rate of this particular structure. More details are presented by Fazio (1999).

The overall project efforts in this study were directed to gain hands-on knowledge and experience with the in-situ durability tests. The results are consistent and these in-situ testing techniques are promising, and the main challenge is for practicing engineers to work in tandem with those involved in relevant research. Thus, practicing engineers, and especially those in the industry involved in testing, are encouraged to increase the use of in-situ testing for durability so as to build up an essential pool of knowledge and experience. This will ultimately lead to the realization of the full potential for in-situ test methods.

5 Acknowledgements

The authors would like to express their deep appreciation of the strategic research grant from the National Sciences and Engineering Research Council of Canada. The cooperation and assistance of the City of Montreal, Corexco Inc., and CERIU are gratefully acknowledged.

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27 DURABILITY OF CRACKED FIBRE REINFORCED CONCRETE STRUCTURES EXPOSED TO CHLORIDES

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Abstract

Durability studies are carried out by subjecting FRC-beams to combined mechanical and environmental load. Mechanical load is obtained by exposing beams to 4-point bending until a predefined crack width is reached, using a newly developed test setup. Exposure to a concentrated chloride solution is used as environmental load. The chloride penetration is characterized both qualitatively (UV-test) quantitatively (chloride profile) and by microscopy. The test programme involves three different concrete qualities. Both steel fibres and polypropylene fibres are used in the concrete beams as well as main reinforcement. The effect of the cracks, the fibres and the concrete quality on the chloride penetration is studied.

Keywords: Chloride exposure, combined mechanical and environmental load, concrete, cracks, durability, fibre reinforcement (PP-fibres, steel fibres)

1 Introduction

The ability of fibres to arrest cracks and to minimize the extent of cracking and crack width is the main subject for the research project "Design Methods for Fibre Reinforced Concrete (FRC)". The change of crack pattern is expected to improve the durability regarding environmental load of FRC-structures compared with plain concrete. An overview of the research programme is given in previous work (Hansen and Hansen 1996).

In the laboratory, different degradation mechanisms are normally treated separately and in most cases no mechanical load is involved when materials are tested

for durability. However, natural exposure of concrete structures consists of a combination of mechanical and environmental load (frost, chloride etc.). Therefore, in this project, specimens are subjected to a combined load.

This paper describes chloride exposure tests. The mechanical load is applied by 4-point bending until a predefined crack width is reached. The environmental load is applied by exposing the specimens to a concentrated chloride solution. The effect of fibres and cracks on the chloride penetration is studied by means of UV-test and Rapid Chloride Test as described in Section 3.2 and 3.3. The project also includes microscopical analysis. These studies are in progress.

2 Materials

Three different concrete mixes relating to three different environmental classes are tested cf. Table 1. M = moderate, A = aggressive, SA = more severe than A. Cement is ASTM Type III for class M or Type V classes for A and SA, aggregates are rounded (class M) or crushed rock. All concretes are air entrained^{1,2}. Fixed amount of two different types of fibres are used, 1.0 vol% polypropylene fibres (PP, length 12 mm) or 0.4 % vol steel fibres (ZP, length 30 mm with hooked ends). When adding fibres to the concrete mix the same volume of the largest aggregates is removed. The following combinations of concrete mixes and fibres are studied: M-0, M-ZP, A-ZP, SA-0, SA-PP, SA-ZP. 0 is referring to concrete without fibres.

Table 1: Concrete mixes without fibres

Materials (kg/m ³)	Class M	Class A	Class SA
Water-powder ratio	0.52	0.44	0.38
Cement	240	298	285
Fly Ash	65	65	60
Silica fume - slurry	16	16	24
Water	142	140	115
Fine aggregate 0-4 mm	774	702	758
Coarse aggregate 4-8 mm	319	340	-
Coarse aggregate 8-16 mm	649	680	535
Coarse aggregate 16-25 mm	-	-	565

¹ 2.5 - 4% air for class SA concrete, 6 - 9% air for classes M and A concrete.

² All concretes contain different amounts of plasticizers to maintain acceptable workability

3 Test setup

Reinforced beams with dimensions 100x200x1150 mm are used as specimens. A test setup has been developed to permit beams to be subjected to combined mechanical and environmental load by means of threaded bars and nuts as described in previous work (Hansen and Nielsen 1997).

3.1 Exposure conditions

Mechanically loaded concrete beams are placed in a 16% NaCl solution. Once a week the beams are allowed to dry out partially for 4 days. A wetting-drying cycle and a concentrated chloride solution accelerates the chloride penetration compared to a permanent immersion and accentuates differences between the concretes as well.

Chlorides may be present in concrete in different forms of bonding. Generally chlorides are classified as freely movable dissolved in the pore-water, physically adsorbed or chemically bound e.g. in the form of Friedel's salt. Normally the amount of free and bound chlorides are determined simultaneously; i.e. the total amount of chlorides. This value can be determined accurately, e.g. as described in Section 3.3, but gives very little information on the distribution of the free chlorides especially in the contact zone with the main reinforcement, which is decisive for corrosion. Therefore, the method described in Section 3.2 is applied.

3.2 UV-test

The chloride penetration is visualized by means of the UV-test (Schöppel et al 1988). A silver nitrate solution is sprayed as a very fine aerosol onto a sawn concrete surface perpendicular to the chloride exposed surface. A corrosion resistant spray gun for compressed air is used. The sawn surface is then exposed to ultraviolet radiation (UV-radiation). After a few minutes of UV-exposure, areas containing free chlorides change colour to blue or grey. Areas without free chlorides remain brown in colour. Previous experiences show that the boundary between blue/grey and brown areas relate to a content of free chlorides of about 0.1% by weight of concrete, depending on the type of cement, etc. The UV-test can indicate the risk of corrosion since concrete structures need repair when spots of corrosion are seen and the main reinforcement is placed in the zone that changed colour into blue or grey. The UV-test is applied on all the exposed beams.

The main reinforcement and the steel fibres are inspected for signs of corrosion.

3.3 Chloride profile

The determination of a chloride profile consists of two steps: 1) collecting samples, 2) determination of chloride content in each sample. In this project, the test samples are produced by milling 12 mm wide grooves across the concrete beams in the tension side. Each time, 5-8 mm of concrete is milled off until the main reinforcement is reached at about 35 mm from the surface. This gives a reasonable volume of each sample, although the chloride profile will be rather rough. Three sets of grooves are made, one directly "in" the crack, the second along the first one and the third along the second, i.e. the middle of the third groove will be 24 mm from the

center of the crack. Germann's Rapid Chloride Test (RCT) is applied for determination of the total chloride content (Germann Instruments 1998). The milled, pulverized samples are placed in a special acid that extracts the chlorides. The amount of acid soluble chloride - expressed as percent weight of concrete - is determined directly by a calibrated chloride sensitive electrode connected to an electrometer. RCT is not applied on class M beams since these are not designed to be exposed for chlorides.

4 Results

4.1 Crack widths and load level

Table 2 shows crack widths and load level for individual beams, shortly after applying the load. The load level is normalized in relation to the load level of concrete SA-0 (2 kNm in the center part). The load level decreases about 30% during the exposure period due to relaxation. No measures are taken to retain the load level.

4.2 Chloride penetration

Figures 1 and 2 show sections of chloride exposed beams after applying the UV-test; the tension zone is on the top of the figures. The boundary between areas with and without free chlorides is marked as well as the mechanically induced cracks. The height of the beams is 100 mm. Figures 3 to 9 show examples of chloride profiles (RCT). Each figure includes three profiles, relating to three sets of milled grooves according to Section 3.3. These are marked by "0 mm", "12 mm" or "24 mm" relating to the distance of the center of the groove from the crack.

Depth of penetration in relation to the UV-test describes the distance from the exposed outer surface where a change in colour can be seen after spraying with silver nitrate. Depths of penetration are summarized in Table 2, around cracks as well as in uncracked area, both in the compression and the tension zone of the beams.

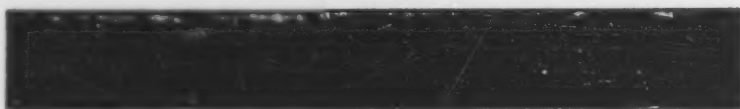


Fig. 1: Section of beam SA-ZP 03 after UV-test. 13 months of exposure. Chloride profile in Fig. 9 is made at the second crack from the left



Fig. 2: Section of beam A-ZP 01 after UV-test. 13 months of exposure. Chloride profile in Fig. 4 is made at the second crack from the right

Table 2: Crack widths and load level. Depth of penetration from exposed outer surface and from crack surface, UV-test

Concrete	Beam ID	No. of cracks	Average crack width w (mm)	Load Level (Normalized to SA-O) ¹	Depth of chloride penetration (mm)	Depth of chloride penetration from exposed outer surface	Typical width of "tongue" (mm) ³
Around a crack in tension zone ²							
Uncracked area of tension zone							
Compression zone							
M-0	03	5	0.27	1	50-100	10-15	25-30
	06 ⁴	3	0.11	0.95	40-50	10-15	15
M-ZP	05	7	0.18	1.2	40-80	20-25	25
	03 ⁴	6	0.12	0.95	60-70	10-15	15
A-ZP	01	7	0.19	1.2	50-80	10-15	25-30
	06 ⁴	5	0.06	0.95	30-50	10-12	10-20
SA-0	01	5	0.27	1	50-70	10-15	15-20
	06 ⁴	3	0.11	1	25-40	5-7	<10
SA-PP	01	7	0.14	1.8	50-75	15-25	15-20
	06 ⁴	7	0.10	1.35	20-40	10-15	<10
SA-ZP	03	7	0.17	1.8	40-70	10-15	10-15
	06 ⁴	4	0.09	1.35	20-50	7-10	<10

1. Load level of SA-O = 2 kN/m

2. Depth of penetration varies with crack depth

3. The width of the chloride "tongue" around a crack in the tension zone of the beam. The width is measured at the root of the tongue. Depth of penetration from the crack surface = $\frac{1}{2} \times$ "tongue" width.

4. Exposure period 7 months; otherwise 13 months

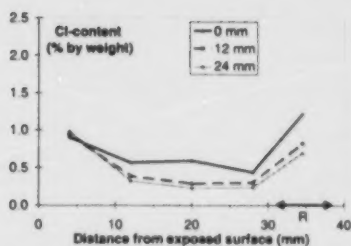


Fig. 3: Chloride profile. Specimen A-ZP 06, $w = 0.06$ mm. 7 months of exposure. R refers to the position of the main reinforcement

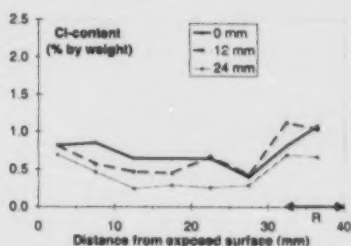


Fig. 4: Chloride profile. Specimen A-ZP 01, $w = 0.16$ mm. 13 months of exposure

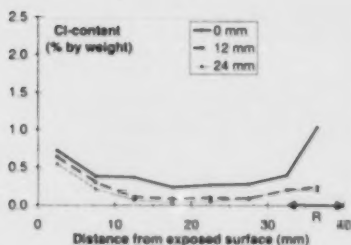


Fig. 5: Chloride profile. Specimen SA-0 06, $w = 0.09$ mm. 7 months of exposure

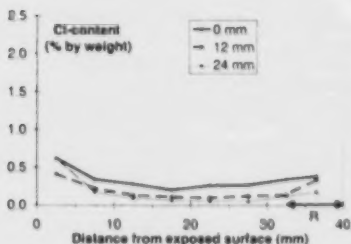


Fig. 6: Chloride profile. Specimen SA-PP 06, $w = 0.14$ mm. 7 months of exposure

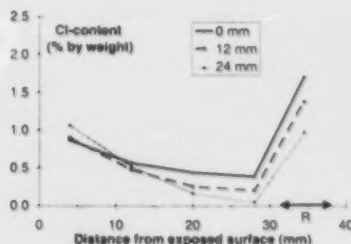


Fig. 7: Chloride profile. Specimen SA-PP 01, $w = 0.20$ mm. 13 months of exposure

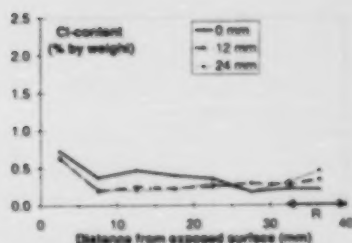


Fig. 8: Chloride profile. Specimen SA-ZP 06, $w = 0.06$ mm, 7 months of exposure. R refers to the position of the main reinforcement

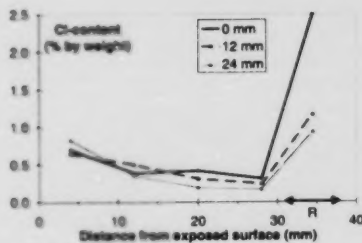


Fig. 9: Chloride profile. Specimen SA-ZP 03, $w = 0.22$ mm, 13 months of exposure

5 Discussion

5.1 Crack widths and load level

The main purpose of adding fibres to concrete is to minimize the crack width and the extent of cracking. Both of these effects are seen, especially on concrete SA where the fibre reinforced beams are loaded 40-80% higher than beams without fibres to reach the same level of crack widths for a beam. The number of cracks is also higher in FRC-beams, i.e. the fibres act as crack dividers.

5.2 Depth of chloride penetration (UV-test)

The depth of the chloride penetration is considerably higher in the zone around a crack compared with uncracked areas and depends on the crack depth, no matter the crack width. The importance of a crack therefore is expressed by its depth rather than by its width. If the depth of penetration around a crack exceeds the concrete cover, the risk of corrosion of the main reinforcement is present, regarding the fact that the UV-test has shown to indicate the risk of corrosion, cf. Section 3.2.

Chloride penetration perpendicular to the crack is increased by the crack width, cf. Table 2. This is probably related to the fact that the specimens with large crack widths have been exposed for a longer time than the ones with smaller crack widths.

The depth of penetration in areas without visible cracks is lowest in specimens of SA-0 and SA-ZP from the exposed outer surface as well as perpendicular to the cracks, indicating that these concretes are the most dense. SA-PP has a larger depth of penetration from the exposed outer surface than SA-0 and SA-ZP, which can be explained by problems in casting this type of concrete. Therefore, the outer layer is of lower quality. Furthermore, SA-PP is loaded to 50-100 % heavier load than SA-0, Table 2. Load level and concrete quality, i.e. water-cement ratio, can also explain differences in depth of penetration between SA-PP, A-ZP, M-ZP and M-0.

The importance of the load level and the quality of concrete on the chloride

penetration corresponds with (Konin et al. 1998) who exposed specimens to a salt fog after they were subjected to uniaxial tension until a crack appeared. Also the density of microcracking was shown to have a significant effect on the chloride permeability.

In areas without visible cracks the depth of penetration is larger in the tension zone than in the compression zone of the beam, especially in specimens with large crack widths and a long exposure period, cf. Table 2. This indicates the existence of microcracks in the tension zone as a result of the mechanical load, which is supported by the fact that no difference in depth of penetration is seen on specimens exposed without mechanical load. The amount of microcracks is currently studied by means of microscopical analysis.

5.3 Chloride profiles (RCT)

In most of the investigated concretes a high content of chloride ions is measured in the outer layer by the Rapid Chloride Test (RCT). Furthermore, a high content is measured at the main reinforcement in the tension side, in several cases higher than in the surface layer. Although some corrosion is seen on the main reinforcement, especially in the compression zone where the concrete cover is only 12-15 mm, no damages are observed in the concrete after 13 months of concentrated chloride exposure. Steel fibres are only corroding if they are in direct contact with the exposed outer surface or a crack surface.

In general, the chloride content in the groove milled "in" the crack - marked by "0 mm" in Figure 3-9 - is almost the same after 7 and 13 months, i.e. a kind of equilibrium is established. The content of chlorides decreases with the distance perpendicular to the crack surface indicating that the cracks act as an extra surface. After 13 months of exposure the content of chlorides 24 mm from the crack normally is twice as high than after 7 months; 0.1-0.4 % by weight, compared to 0.05-0.2 % by weight.

In the following, the high amount of chlorides at a depth corresponding to the concrete cover is discussed. While the water in the cracks evaporates during the drying part of the drying-rewetting cycle, the chlorides are left behind, thereby creating a concentration gradient between the outer part and the inner, still-water-filled part of the crack. To balance out the gradient, there will be a transport of chlorides inwards. Thereby, the concentration of the chloride content increases until the solution eventually becomes saturated with chloride and the salt begins to crystallize.

A crack width at the surface of 0.2 mm, a crack depth of 40 mm, a triangular shape of the crack and a concentration of the NaCl solution of 16% is assumed. Based on volumetric calculations and the concentration of NaCl solution at crystallization, crystallization then will start when the water front is 10 mm from the exposed surface. On the other hand, the measured chloride profiles indicate that crystallization does not take part until 30-35 mm from the exposed surface, cf. the very high chloride content at this depth. The somewhat simplified calculation could explain this discrepancy, e.g. the calculation assumes that the chloride concentration in the crack equals the concentration in the solution surrounding the concrete specimen when the concrete starts to dry out.

Studies by (Sakai and Sasaki 1994) supports the observations of a high chloride content inside the crack. Mechanically loaded beams exposed to chlorides at a coastal

exposure site for several years showed that 0.1 mm wide cracks contained more chlorides around the main reinforcement than 0.2 mm wide cracks. It was concluded that the more narrow the crack the more difficult it is to wash out the chlorides.

In (Reinhardt et al. 1998), mechanical tests with notched specimens are described where cracks are created by means of a wedge splitting test or a bending test. It is concluded that cracks smaller than 0.03 mm do not influence the penetration of liquids, i.e. that the concrete in the tension zone behaves as it is without cracks. On the other hand (Eriksen et al 1996) finds that, even at a crack width of 0.01 mm, a marked by higher chloride concentration is observed around cracks compared with uncracked concrete. Differences in concrete quality and exposure conditions can explain this discrepancy.

5.4 Comparison of UV-test results and chloride profiles

Qualitatively, there is a connection between the results from the UV-test and the chloride profiles. In general, there is a high chloride content in the groove milled directly "in" the crack. After 7 months there is no difference between the chloride content at 12 and 24 mm from the crack surface, while the content is higher at 12 mm after 13 months. This corresponds to the fact that 1) the width of the chloride tongue is largest for the beams exposed in 13 months, 2) the width of the chloride tongue after 7 months of exposure equals the width of the milled groove.

Since the UV-test gives an indication of the amount of free chloride ions, while the chloride profile expresses the total amount of chloride, the numerical values based on the two methods are not comparable. According to (Neville 1995), the proportion of bound chloride ions varies from 80% to well below 50% of the total chloride ion content. In the milled grooves where the middle of the groove is 24 mm from the crack, a chloride content of 0.1-0.4 % by weight is reached. Assuming that about 2/3 of the chloride content measured by the RCT are bound, there are less than 0.13% free chlorides. This supports the previously reported indication level of the UV-test (0.1% by weight) since the width of the chloride tongue indicates that the groove milled 24 mm from the crack surface is "outside" the chloride tongue, cf. Table 2.

6 Conclusions

The existence of cracks affects the chloride penetration heavily; the depth of penetration is considerably higher along cracks when the cracks are 0.05-0.35 mm wide than in uncracked concrete. This is illustrated by the UV-tests. It has not been possible to confirm investigations indicating the existence of a lower limit of crack width below which no chloride penetration is observed. Differences in depth of chloride penetration between tension and compression zones of the specimen indicate formation of microcracks in the tension zone. Microscopical analysis is in progress to confirm this and to further investigate the chloride distribution in cracks and microcracks.

The result of the UV-tests indicate that the crack depth, the concrete quality and the duration of the exposure is more important for the chloride penetration than the crack width. Crack depths corresponding to the concrete cover are especially problematic, referring to the risk of corrosion of main reinforcement. A very high

content of chloride at the main reinforcement indicates crystallization of chlorides.

The effect of the fibres on the durability increases with the concrete quality. To reach the same level of crack width and depth of chloride penetration, the load must be 40-80 % higher for class SA concrete when fibres are added, but only 10-15 % for class M concrete. Steel fibres are only corroding if they are in direct contact with the exposed surface or a crack surface.

7 Acknowledgement

The present work is part of the research project *Design Methods for Fibre Reinforced Concrete* funded by the Materials Technology Development Programme, MUP2, under the supervision of the Danish Council of Technology, the Danish Technical Research Council and the Danish Natural Science Research Council.

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28 EDS ANALYSIS AS A PREDICTION METHOD FOR CEMENT DEGRADATION IN AN AGGRESSIVE ENVIRONMENT

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Abstract

The behaviour of Portland cement with and without 30% fly ash addition in aggressive solutions was studied. This paper considers the sulphate corrosion of the hardened cement paste. The cement paste test samples were prepared, cured in water for 21 days and then exposed to the influence of aggressive environment (10% $(\text{NH}_4)_2\text{SO}_4$ solution). The measurements of several properties were carried out and the results were analyzed. Concentration gradient of sulphate ions in cement paste depth was obtained by energy-dispersive analysis (EDS).

Keywords: Corrosion, EDS, durability, concrete, portland cement, fly ash

1 Introduction

Unless it is resolved successfully, as a part of the integral environmental management, the problem of wastes might become one of the burdens on the future generations. Besides environmental pollution, this also concerns some important economic reasons connected with the growing price of raw materials and energy. That is why the efforts - primarily on behalf of the industrially developed countries - are more and more directed towards recycling and various forms of using wastes. Thermo-power stations, which are using coal as the fuel, mostly have installed very efficient equipment for preventing the emission of solid particles (fly ash) to the atmosphere. For our country it is about 10,000,000 t per year of this material. Wetted material is then transferred to deposit places which presents very big ecological problem. By definition, fly ash is a fine powder of mainly spherical, glassy particles having pozzolanic properties and consisting essentially of SiO_2 and Al_2O_3 . Because of that, the aim of our first investigations was to establish procedure for deposits of waste fly and bottom ash utilization in

building materials production. Reaching this aim means reducing enormous ecology problem so as the getting equal quality products (cement, mortar, concrete, bricks, etc.) (Biczok 1972; Mehta 1983).

In practise, concrete buildings suffer simultaneously mechanical, chemical and physical attacks. Therefore, the effect of mechanical stresses must be taken into consideration when durability and corrosion resistance of a concrete are estimated, i.e., the study of stress corrosion of concrete is necessary and very important for durability (Moskvin 1980; Regourd 1980).

In recent years, there has been an increasing need to develop new testing methods for monitoring the sulphate attack on concrete structures. There are many test methods available for the assessment of the quality and degradation of concrete structures. The basic idea of this work was to test the possibility for applying energy dispersive analysis in our case. The aim was to investigate sulphate attack on concrete structures, i.e. to obtain concentration gradients of sulphate ions through the sample of cement paste. This was done by monitoring concentration of elementary sulphur through cross sections of the sample, in line, because EDS is a technique which determines the elemental content in a very small surface area. The advantages of this method are that it is non destructive and does not require any additional sample preparation.

2 Experimental

Portland cement and Portland fly ash cement manufactured in Yugoslavia were used:

- Portland cement B (PCB)- according to the European cement standard EN 197-1: CEM-I
- Portland fly ash cement B (cement clinker B) with 30% fly ash (PCBP)- according to the European cement standard EN 197-1: CEM II/B-V

All materials have been chemically analyzed and their physico-chemical and mechanical properties have been investigated. The resistance of the cement to the aggressive solution was tested on mortar prisms. First results were already published (Miletic et al. 1996, 1997). The testing methods used in this part are: determination of standard strength (EN 196-1), chemical analysis (EN 196-2), determination of setting time (EN 196-3), determination of the sieve residue (EN 196-6), determination of specific surface (EN 196-6), calculating the potential phase analysis (ASTM C 150).

To analyse the degradation process caused by sulphate attack, samples were prepared as follow. The chemical resistance of the cements was tested with cement paste 1x1x6 cm test prisms prepared with water/cement ratio needed for the standard consistency. Before beeing exposed to the aggressive solution, prisms were cured for 1 day in the mould and 20 days in water. As an aggressive environment, a 10 % $(\text{NH}_4)_2\text{SO}_4$ solution was used.

The concentration gradient of sulphate ions in prepared cement paste samples, dimensions 1x1x6 cm and broken on halves (1x1x3 cm approximately), was performed by an energy-dispersive method on an Edax 9900 apparatus at the

cross-section of the broken sample. For this purpose, samples were mounted on stubs and pumped to a vacuum level of $1.33 \cdot 10^{-6}$ Pa. Monitoring of elementary sulphur content was done in 20 points on 1 cm width of samples, so the monitoring on the centered line of the cross-section was obtained.

3 Results and discussion

The selected aggressive environment was severe to ensure fast results for the real conditions which can be present in underground waters in Yugoslavia.

The potential phase analysis of the Portland cement clinker is given in Table 1. It can be seen that the cement has usual C_3A content in clinker of 13.3% influencing the sulphate resistance. The ordinary Portland cement, such as the investigated one, is not resistant to the attack of sulphates because it has a considerable content of tricalcium aluminate - C_3A , whose hydrates react with sulphate ions, giving expansive compounds. According to the literature the difference in the C_3S content could be significant regarding sulphate resistance also.

Table 1: Potential phase composition of Portland cement clinker

Potential phase composition, % mass	Portland cement clinker
	B
C_3S	57.5
C_2S	13.5
C_3A	13.3
C_4AF	8.7

Table 2 presents the chemical composition of fly ash. According to the high content of SiO_2 , Al_2O_3 and Fe_2O_3 and the low content of CaO , MgO , alkalies and SO_3 the fly ash is suitable for cement production though loss on ignition was relatively high and especially hydrated (bounded) water.

Table 2: Fly ash chemical composition

Chemical composition, % mass	Fly ash
LOI	5.7
$SiO_2 + Al_2O_3 + Fe_2O_3$	84.2
CaO	6.5
MgO	2.7
SO_3	0.05
S	0.02
Na_2O	0.3
K_2O	0.7
Hydrated water	34.6
Insoluble residue	76.6

The chemical composition of the cements is presented in Table 3. All cements meet Yugoslav standard JUS B.C1.011. Portland fly ash cement PCBP has a higher insoluble residue and loss on ignition than Portland cement PCB.

Table 4 presents figures characterizing fineness, density, standard consistency, setting time and soundness of the test cements. Obviously the addition of fly ash raises the water demand for standard consistency and sieve residue and extends setting time, but has no significant influence on other characteristics. All characteristics are in compliance with Yugoslav standard JUS B.C1.011.

Table 3: Chemical composition of cements

Chemical composition, % mass	Cement	
	PCB	PCBP
SiO ₂	19.7	14.0
Al ₂ O ₃	7.0	6.2
Fe ₂ O ₃	2.7	2.7
CaO	62.0	44.7
Insoluble residue	0.1	20.2
LOI	0.8	3.0
CO ₂ in CaCO ₃	0.4	0.4
CaO free	0.1	0.0
SO ₃ in CaSO ₄	2.0	2.0
MgO	2.2	2.4
Alkalies as Na ₂ O	0.4	0.4
K ₂ O	0.4	0.2
Cl ⁻	0.0	0.0

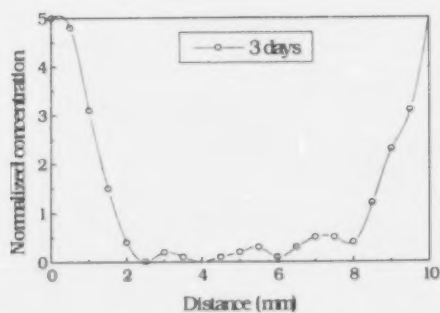
Table 4: Physico-chemical properties of cements

Physico-chemical properties	Cement	
	PCB	PCBP
Sieve residue at 0.09 mm sieve, %mass	1.8	5.2
Density, g/cm ³	3.1	2.9
Specific surface, cm ² /g	3320	3720
Setting		
-standard consistence, %mass	25.8	28.0
-initial time, min	165	240
-final time, min	225	330
Soundness		
-Le Chatelier test, mm	1.0	1.0

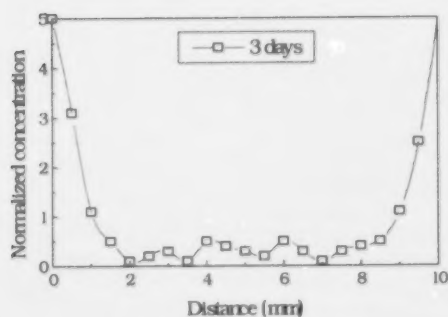
Table 5 gives values for flexural and compressive strengths of cements after 2, 3, 7 and 28 days. The Portland fly ash cement PCBP had lower strengths even after 28 days than the corresponding Portland cement PCB.

Table 5. Standard strength of cements

Strengths, MPa	Cement	
	PCB	PCBP
Flexural:		
-2 days	4.4	2.5
-3 days	5.3	3.6
-7 days	7.2	6.2
-28 days	8.0	8.3
Compressive:		
-2 days	15.7	8.8
-3 days	19.8	14.9
-7 days	30.2	24.2
-28 days	40.3	39.5



(a)

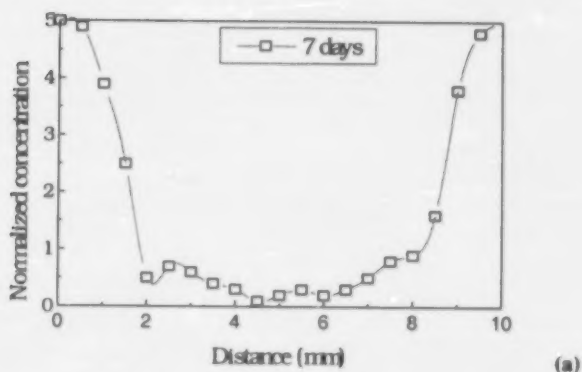


(b)

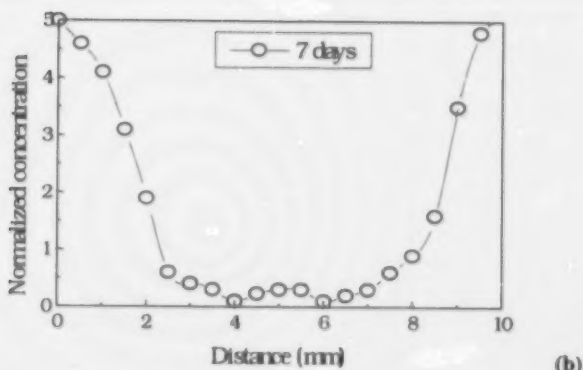
Fig. 1: Concentration gradient of sulphur through the depth of sample after 3 days in aggressive environment, determined by EDS method, for:

a) Portland cement PCB

b) Portland ash cement PCBP

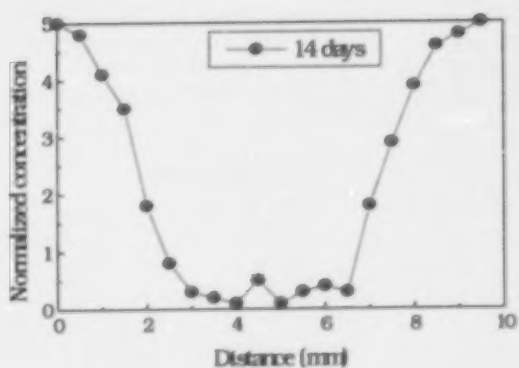


(a)

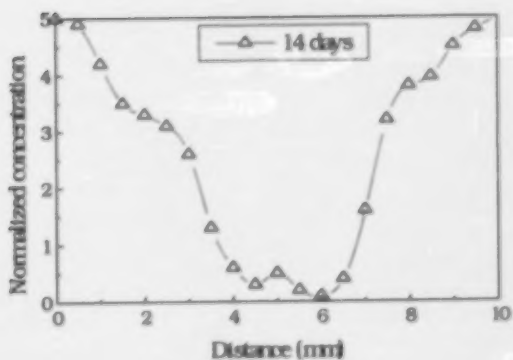


(b)

Fig. 2: Concentration gradient of sulphur through the depth of sample after 7 days in aggressive environment, determined by EDS method, for:
 a) Portland cement PCB
 b) Portland ash cement PCBP

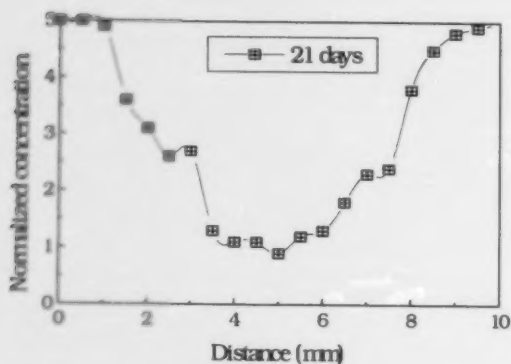


(a)

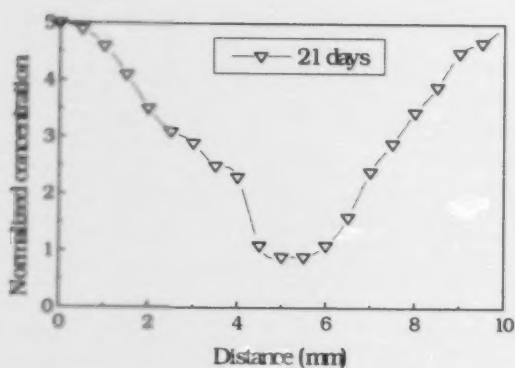


(b)

Fig. 3: Concentration gradient of sulphur through the depth of sample after 14 days in aggressive environment, determined by EDS method, for:
a) Portland cement PCB
b) Portland ash cement PCBP



(a)



(b)

Fig. 4: Concentration gradient of sulphur through the depth of sample after 21 days in aggressive environment, determined by EDS method, for:
a) Portland cement PCB
b) Portland ash cement PCBP

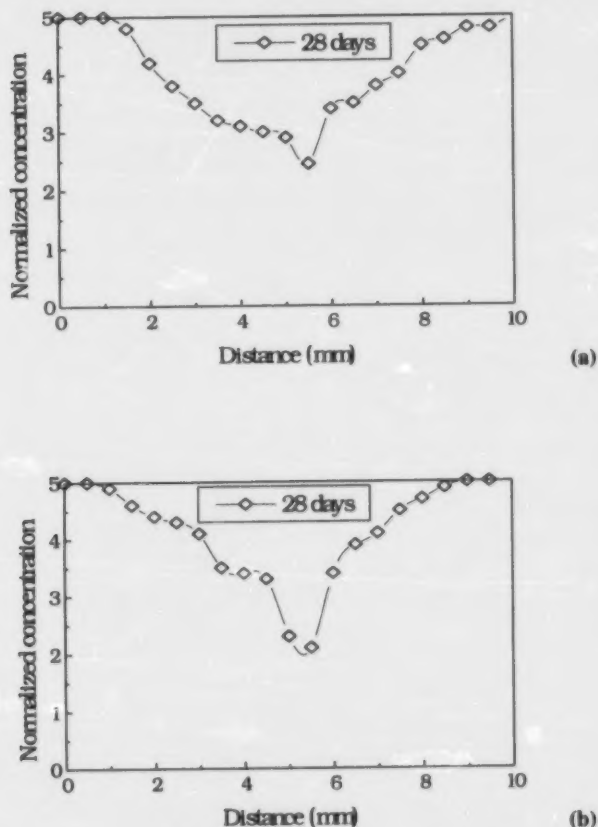


Fig. 5: Concentration gradient of sulphur through the depth of sample after 28 days in aggressive environment, determined by EDS method, for:
 a) Portland cement PCB
 b) Portland ash cement PCBP

The content of sulphur, i.e. content of sulphate, as determined by the EDS method, is given in Figs. 1-5. The distance, given on X-axis, means the distance from one side of the cross-section of the broken half to the other one (10 mm at all) through the centered line of the cross-section. It is important to emphasise that, comparing two tested cements, all obtained very similar curves. This means that the behaviour of both cements in the aggressive environment is similar. But, regarding maximum normalized value, 5, it could be seen that Portland fly ash cement PCBP has less points at that level in all terms of investigation, meaning

that sulphate content through depth of test sample is lower. The repeated testings showed the same results, so it could be concluded that this method has the high accuracy. Such results were got also by previous investigations (Miletic et al. 1996, 1997) meaning that Portland fly ash cement PCBP is more suitable for use in aggressive environment than the ordinary Portland cement PCB.

4 Conclusion

The results of testing the attack by aggressive sulphate solutions allow the following conclusions:

- 1) The selected aggressive environment represents a very strong aggressiveness to ensure fast results for the real conditions which can be present in underground waters in Yugoslavia.
- 2) Testing results showed that EDS himself couldn't represent the degradation process, but combined with other significant characteristics such as mass change, XRD, bonded SO_4^{2-} content and flexural strengths (degradation and/or corrosion coefficients derived from them), represented valuable characteristics.

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29 PREDICTION OF CONCRETE CRACK OPENING CAUSED BY REINFORCING BAR CORROSION

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Abstract

The influence of reinforcing bar corrosion on the crack opening width of reinforced concrete surface was investigated. A fracture mechanics based numerical analysis method was developed in order to predict the crack width on the concrete surface due to the expansive force of the corroded reinforcing bar. The corrosion accelerating tests for reinforced concrete model specimens were performed on the various kinds of concrete, and relationships of crack opening and the degree of corrosion were clarified. The crack width obtained from the accelerated test agreed well with the results of calculations from the numerical analysis by considering the creep effect and the tension softening properties of concrete.

Keywords: Crack, corrosion, reinforcing bar, fracture mechanics, tension softening

1 Introduction

In order to increase the durability of existing reinforced concrete structures, it is essential to grasp the deterioration phenomena of such structures and select suitable repair methods. Critical deterioration of reinforced concrete structures can be explained as a phenomenon in which "corrosion of reinforcing steel or reaction of aggregate causes cover concrete to be subjected to expansive or tensile action, and the resulting cracking with concomitant acceleration of reinforcement corrosion reduces the strength of reinforcing bars, lowering the structural yield capacity of the member." In other words, it is important in terms of durability for concrete to be sound against expansive and tensile forces inducing corrosion. Also, carbonation and salt damage

are regarded as factors (internal deterioration forces) for causing expansive forces of reinforcing bars. It is fundamental to the study of concrete durability to elucidate the process of failure of cover concrete subjected to such deteriorating internal forces and find a method of preventing such failure.

Though a number of studies have been conducted regarding reinforcement corrosion, there have been few studies on the changes in the crack width and propagation of a fracture process zone after the onset of cracking on the concrete surface. Analysis by fracture mechanics incorporating the softening properties of concrete is considered particularly effective in analyzing crack propagation of nonlinear materials, such as cementitious materials. In this study, the authors grasp crack opening on the surfaces of various concretes resulting from corrosion-induced expansion of reinforcing steel by electrolytic corrosion experiments and propose an analysis method based on nonlinear fracture mechanics using cohesion force models. Comparisons are made between analyzed and tested results. A method of estimating the cross-sectional loss of reinforcing bars from the crack opening on the concrete surface is discussed as well.

2 Experimental procedure

2.1 Materials and specimens

Five types of mortars/concretes are used: normal strength plain mortar (NPL), normal strength fiber reinforced mortar (NVF), high strength plain mortar (HPL), high strength fiber-reinforced mortar (HVF), and normal concrete (Ncon). Fiber-reinforced specimens are included to represent those with particularly high tensile toughness. The mix proportions are given in Table 1. The cement is high flow cement; the fine aggregate is manufactured sand from Hachioji; and the coarse aggregate is crushed quartz from Danto. Silica fume and super plastisizer are used as the mineral and chemical admixture, respectively. The fibers are made of Vynlon with a specific gravity of 1.3, length of 30 mm, and strength of 90 kgf/mm². The 28-day compressive strength and tensile strength of the specimens are given in Table 1. Fig. 1-a) shows the geometry of the specimens. Prismatic specimens 100 by 100 by 150 mm in size are used, in which round bars 20 mm in diameter are embedded with specified cover depths by using a processed mold. The cover depths are basically 5, 15, and 30 mm to include the situation where insufficient cover causes durability

Table 1: Mix proportion

Specimen	sign	W/B	water	cement	SF	sand	gravel	SP	fiber	tensile strength MPa	compressive strength MPa
		[α]	(kg/m ³)								
Normal plain mortar	NPL	50	250	500	0	1509	-	0	-	7.1	54.9
Normal fiber mortar	NVF	50	243	497	0	1479	-	5.0	17.9	8.4	58.3
High plain mortar	HPL	20	193	964	107	1101	-	21.4	-	8.9	120.0
High fiber mortar	HVF	20	182	964	107	1064	-	32.1	18.6	12.7	123.0
Normal concrete	Ncon	50	176	360	0	774	1040	4.0	-	6.3	54.8

B=SF+cement SF: silica fume SP: super plastisizer

problems. Protruding ends of round bars are coated with epoxy resin and a sealing compound to prevent corrosion. The crack width is measured using two π -gages attached to each specimen. Two specimens are used for each set of conditions.

2.2 Apparatus

The electrolytic corrosion equipment is shown in Fig. 1-b). A situation in which reinforcing steel corrodes uniformly due to, e.g., internal chlorides is simulated in this study. The bottom of a specimen is immersed in salt water (an NaCl 10% solution), and a 30V constant voltage is impressed from a DC regulated power source through the bar, the anode, and a copper plate partly immersed in the salt water, the cathode. Specimens were mostly submerged in salt water in past studies, but this causes rust to exude, leading to errors. For this reason, only the bottom of the specimen is immersed in salt water in this study, and the entire surface of the specimen is covered with a cloth wet with salt water. Also, all of the equipment is sealed to prevent evaporation of salt water. The voltage at the resistance in series connection is recorded using a data logger, to determine the current for estimating the amount of corrosion and calculate the integrated current, I_m (Ah). As the actual amount of corrosion products is difficult to determine accurately, the weight losses are measured to evaluate the losses by corrosion. This was done by removing rust from the bar after the tests and measuring the changes in weight. These are then converted to weight losses per unit area, which are taken as the corrosion loss, w (mg/cm^2). The integrated current and corrosion loss being proportional, their conversion factor, α , is determined by Eq. (1). Table 2 gives the average conversion factors of bars in the specimens.

$$w = \alpha \cdot I_m \quad (1)$$

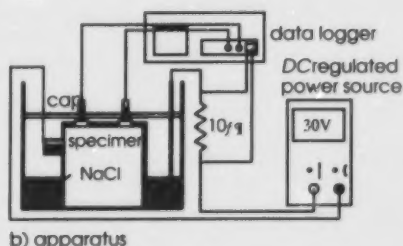
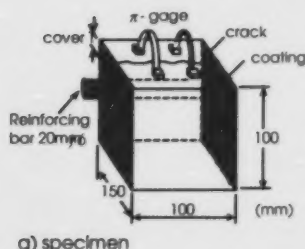


Fig. 1: Outline of test

Table 2: Corrosion data of reinforcing bar

specimen - cover(mm)	weight loss ratio f (%)	corrosion loss w (mg/cm ²)	integrated current Q (Ah)	conversion factor f_c (mg/Ah·cm ²)
NPL-1	6.20	324.8	23.55	13.8
NPL-15	4.01	208.2	20.12	10.2
NPL-30	2.56	132.2	24.69	5.4
NVF-5	5.28	273.1	21.14	12.9
NVF-15	2.78	144.9	19.62	7.4
NVF-30	8.01	412.4	28.80	14.3
HPL-1	4.37	223.7	2.21	101.5
HPL-15	3.28	169.6	3.40	49.9
HVF-15	3.48	179.9	4.77	37.8
Ncon-15	7.03	366.2	14.02	26.1

2.3 Tension softening properties of materials

In addition to compressive strength and Young's modulus, tension softening curves are effective in evaluating the material properties of concretes used for the crack propagation analysis. Three-point bending tests over a span of 400 mm were therefore conducted using a servo-controlled loading tester on notched beams 100 by 100 by 450 mm in size of mortars and concretes under analysis to determine the load-load point displacement curves. These are approximated to tension softening curves by polylinear approximation analysis (Kitsutaka 1997) and then converted to bilinear tension softening curves by least square approximation to facilitate the crack analysis. Fig. 2-a) and 2-b) show the measured load-load point displacement curves and determined tension softening curves, respectively. Though the maximum load and initial bond stress of high strength mortars/concretes are higher than those of normal mortars/concretes, their reductions in the softening zone are greater, showing a tendency towards brittleness. The addition of fibers significantly improved the toughness of both normal and high strength mortars/concretes. Table 3 gives the bilinear parameters for analysis determined by least square approximation. In the table, E_c denotes Young's modulus, f_t denotes initial tensile stress, δ_l and σ_l denote displacement and stress at the elbow point, respectively, and W_c and σ_2 denote critical crack opening displacement and stress at W_c , respectively (see Fig. 3).

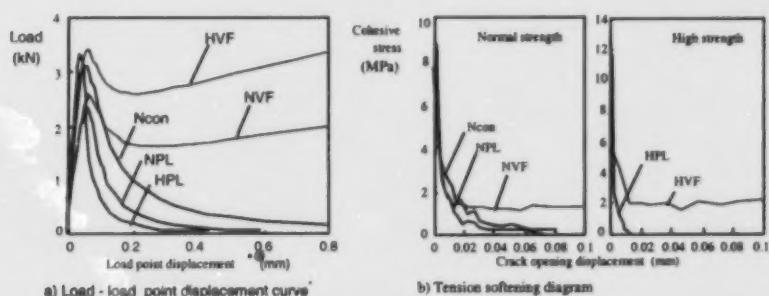


Fig. 2: Analysis of tension softening diagram

Table 3: bilinear parameters

sign	E_c (GPa)	f_t (MPa)	f_A (mm)	f_D (MPa)	w_c (mm)	f_B (MPa)
NPL	33.93	7.91	0.010	0.80	0.072	0.00
NVF	27.26	8.13	0.010	1.10	0.613	2.22
Ncon	42.07	8.96	0.015	1.00	0.280	0.00
HPL	46.77	11.61	0.004	1.80	0.012	0.00
HVF	44.36	7.91	0.012	1.90	0.496	3.12

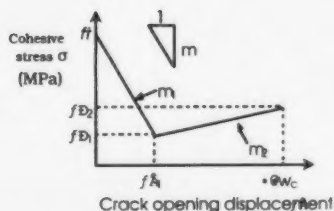


Fig. 3: Bilinear type tension softening curve

3 Results and discussion

The principal cracks occurred along the reinforcing bar upward into cover concrete and downward in the opposite direction of each specimen. The cracks on the surfaces were all within the measurable ranges of π -gages, causing no difficulty in measuring the crack width. In addition to principal cracks, minute cracks developed laterally from the bar, forming a cross of cracks in mortars without fibers. In concretes and fiber-reinforced mortars, minute cracks developed radially from the bar, exhibiting a tendency of dispersing crack-propagating forces. Fig. 4 shows the relationships between the crack mouth opening displacement (CMOD) and the corrosion loss of reinforcement determined by conversion from the integrated current using Eq. (1). The opening displacement for each specimen is the average of two π -gage measurements. The crack width tends to increase as the corrosion loss increases in all cases. High strength mortar/concrete specimens (H) tend to lead to a higher rate of crack width gains after the crack onset than normal strength specimens (N). This may be because cracks propagate in a brittle manner in high strength mortar/concrete after the crack onset, due to its dense microstructure. In normal strength mortar/concrete, the stress is relaxed by minute cracks and plastic deformation around the bar due to its weak microstructure. Though no marked difference is observed in the rate of crack propagation between mortars/concretes with and without fibers after crack onset, fibers delay the crack onset, exhibiting their reinforcing effect. The addition of fibers particularly delays cracking of high strength specimens (HVF-15). In regard to the relationship between the cover depth and the corrosion loss required for cracking, the larger the cover depth, the larger the corrosion loss required for cracking. Cover depth strongly affects the crack onset time.

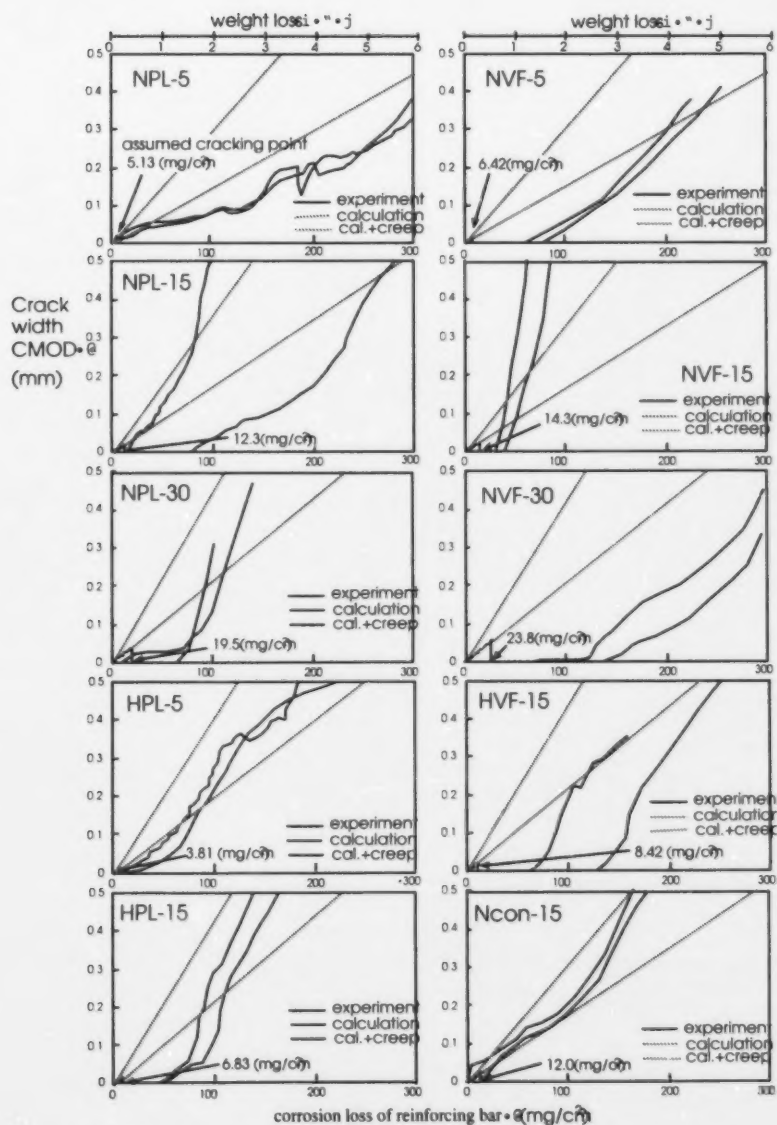


Fig. 4: Relation between crack width and corrosion loss of reinforcing bar

4 Analysis method

Crack opening in Mode I (deformation under tension) induced by reinforcement corrosion is modeled by cohesive force model. As shown in Fig. 5-a) and 5-b), the process is modeled in two phases, i.e., before and after surface crack opening. The stress generated at each node along the crack is assumed to be either the cohesive force, σ , or the extending force, P , equivalent to the expansive force of reinforcement, in consideration of the nonlinear elastic deformation of concrete. Formulation is then made using Eqs. (2) and (3) on the basis of the principle of superposition so that the sum of the stress intensity factors at the tip of the fictitious crack resulting from the stress at each node can be balanced with the crack opening displacement at each node calculated from the stress intensity factor. The stress intensity factor for specimens of a simple shape can be readily referred to in handbooks (Tada et al. 1985) and other sources.

$$K(a) = K_p(a) + K_r(a) : \delta(a, x) = \delta_p(a, x) + \delta_r(a, x) \quad (2, 3)$$

where $K(a)$ = stress intensity factor at the crack tip when the crack length is a , $K_p(a)$ and $K_r(a)$ = stress intensity factors at the crack tip due to equivalent expansive force of reinforcement and cohesive force, respectively. $\delta(a, x)$ = crack opening displacement at x , $\delta_p(a, x)$ and $\delta_r(a, x)$ = crack opening displacements at x due to equivalent expansive force of reinforcement and cohesive force, respectively. The equivalent expansive force of reinforcement is modeled as Eq. (4) by assuming a constant expansive pressure on the bar surface and extracting only the intensity component in the crack opening direction. T , D , and $l(x)$ denote the cover depth, bar

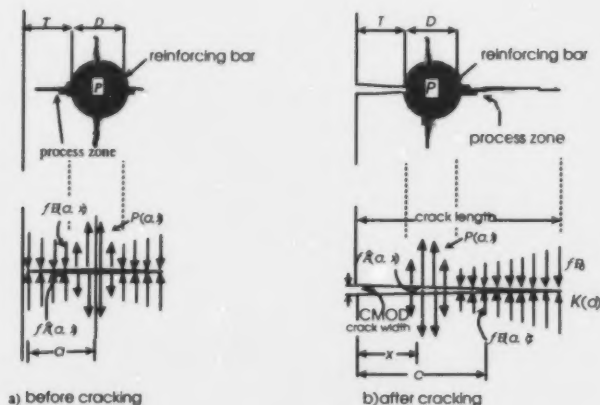


Fig. 5: Modeling of crack propagation by reinforcing bar corrosion

diameter, and component function, respectively.

$$P(a, x) = R_b(a)I(x), \quad I(x) = \sqrt{1 - \left(1 - \frac{2(x-T)}{D}\right)^2}, \quad (T \leq x \leq T+D) \quad (4)$$

The relationship between the cohesive stress (negative values) and the crack opening displacement is expressed using a tension softening curve, which is a material characteristic. A bilinear curve is applied in this study (see Fig. 3).

$$\sigma(a, x) = m(\delta, x) \cdot \delta + n(\delta, x), \quad \delta = \delta(a, x) \quad (5)$$

The cohesive force at the bar position is assumed to be nil. The crack opening displacements can be expressed by the integral equation of stress intensity factors according to Castigliano's theorem. By eliminating $P(a)$ and rearranging the equations Eqs. (2) and (3), integral equation (6) is obtained. $H(a, x, c)$ is a weight function and independent of the acting force.

$$\delta(a, x) = \frac{1}{E} \int_0^a \sigma(a, c) H(a, x, c) dc \quad (6)$$

The distribution of crack opening displacement at a given fictitious crack length of a can be determined by solving nonlinear simultaneous equations obtained by substituting constitutional equations (5) into Eq. (6) and discretizing the equations. The cohesive stress distribution and expansive force can also be determined. Nonlinear simultaneous equations are solved by iteration of the solution until the inclination of constitutional functions of all nodes coincide to determine the optimum solution (Kitsutaka 1997).

5 Analysis results

Fig. 4 shows the analysis results of various corrosion test conditions using bilinear parameters of each specimen given in Table 2. The corrosion loss was calculated from the analysis results of reinforcement expansion (crack opening displacement of cohesive force models). As for rust data, Young's modulus and volumetric expansion of rust were assumed to be 250 MPa and 2.5, respectively, referring to the literature (Sumimoto et al. 1989). An analysis taking account of creep displacement of mortar/concrete surrounding reinforcement was also conducted. In this case, the crack opening displacement at which the expansive pressure on the reinforcement is the same was calculated taking account of the creep coefficient. The creep coefficient was assumed to be 1.0 and 0.5 according to compressive strength for N series (approximately 50 MPa) and H series (approximately 100 MPa), respectively.

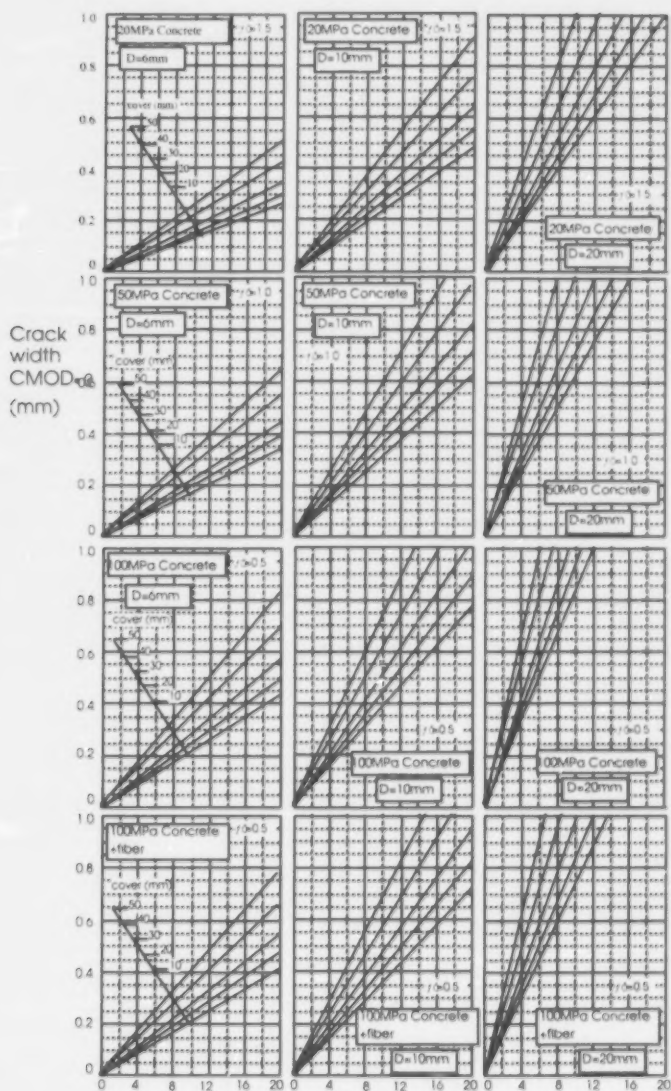


Fig. 6: Relationship between crack width and cross-sectional loss of reinforcement

Fig. 4 reveals that the analysis results taking account of creep generally exhibit good agreement. This may be because the plastic deformation around reinforcement due to expansive pressure can be expressed by creep coefficients. Also, the results agree better for the high strength series than the normal strength series. This may be partly because high strength specimens tend to lead to a single brittle crack, which agrees well with the cohesive force model assuming a single crack, whereas normal strength specimens lead to cracks dispersed around reinforcement.

Fig. 6 shows the relationship between the crack width and the percentage of cross-sectional loss based on this analysis. The diagrams exhibit the relationships for the following conditions: three compressive strengths, 20, 50, and 100 MPa, for which the creep coefficient, f , is assumed to be 1.5, 1.0, and 0.5, respectively, three bar diameters, 6, 10, and 20 mm, and five cover depths, 10, 20, 30, 40, and 50 mm. Using these diagrams, an approximate percentage of a cross-sectional loss can be estimated from the crack width on the surface for each concrete strength, internal bar diameter, and cover depth. The percentage of cross-sectional loss of reinforcement increases as the crack width increases. When the crack widths are the same, a larger cover depth or larger bar diameter indicates a lower percentage of cross-sectional loss.

6 Conclusions

- 1) The relationship between the crack width on concrete surfaces and the corrosion loss of reinforcement was grasped through electrolytic corrosion experiments on specimens in which a reinforcing bar was embedded using normal and high strength mortar/concrete with or without fiber reinforcement.
- 2) A method of analyzing the relationship between crack width and corrosion loss of reinforcement based on fracture mechanics was presented. The integration of creep deformation around reinforcing bars led to relatively good agreement with experiment results.
- 3) The authors proposed diagrams for estimating the cross-sectional losses of internal reinforcing bars from the surface crack width.
- 4) The addition of fibers improved the crack resistance of specimen.

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30 THE USE OF GALVANOSTATIC PULSE MEASUREMENTS TO DETERMINE CORROSION PARAMETERS

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Abstract

This paper reports the results of galvanostatic pulse experiments to determine the corrosion parameters associated with passive and active reinforcing steel in concrete. Galvanostatic pulse measurements have been conducted on a number of short sections of steel bar embedded in concrete. The dynamic response to an applied galvanostatic pulse is measured and used to evaluate the instantaneous rate of corrosion. The bars have displayed a range of corrosion rates, from passive steel to highly active corrosion. Analysis of the galvanostatic pulse response has enabled the separation of components that make up the charge transfer resistance to be determined. By undertaking galvanostatic pulse experiments at a range of lateral distances from the reinforcing bar it has been possible to determine which of the components are associated with the corrosion process and which are due to diffusion effects within the concrete. From these results it is possible to calculate the corrosion rate of the reinforcing steel specimens monitored and to determine suitable equilibration times for linear polarisation resistance measurements for reinforcing steel in concrete in general.

Keywords : Concrete, Corrosion Rates, Galvanostatic Pulse, Steel Reinforcement, Structural Assessment, Dynamic Response

1 Introduction

The problem of accurately and rapidly assessing the condition of corroding steel in reinforced concrete structures has long been a problem for the Civil Engineering industry. A number of electrochemical techniques have been developed to assess the corrosion equilibrium and corrosion rate of the reinforcing steel and to enable an estimate of the service life remaining to a reinforced concrete structure.

The most established of these techniques is half-cell potential mapping. The steel potentials relative to a stable reference half cell are recorded and are related in the guidelines given in ASTM C876-91 to the probabilities of corrosion occurring. However, these are only broad guidelines and the different environmental conditions in individual structures can significantly effect the likelihood of finding corrosion.

Other techniques have been developed to give a direct measure of the instantaneous corrosion rate and include the Linear Polarisation Resistance (LPR) (Gowers et al., 1992) and AC Impedance methods (John et al. 1991). The typical classification of the corrosion rates is given in Table 1.

Table 1: Typical corrosion rates

Rate	R_{ct} ($\Omega \cdot \text{cm}^2$)	i_{cor} (μAcm^{-2})
High	0.25 - 2.5	10 - 100
Medium	2.5 - 25	1 - 10
Low	25 - 250	0.1 - 1
Passive	above 250	less than 0.1

LPR has been developed based on Stern-Geary theory where the corrosion current, I_{cor} , is given by:

$$I_{cor} = \frac{\beta_a \beta_c}{2.3 (\beta_a + \beta_c)} \cdot \frac{1}{R_{ct}} = \frac{B}{R_{ct}} \quad (1)$$

β_a , β_c - Tafel constants; B - Stern-Geary Constant; R_{ct} - Charge Transfer Resistance.

To calculate the R_{ct} value the reinforcing steel is polarised from its equilibrium potential by a fixed amount. This is usually in the range 10 or 20 mV to ensure that for active corrosion the shift lies within the linear region of the Stern-Geary plot. The resulting current is then monitored at the end of a selected time period, usually between 30 seconds to 5 minutes. At the end of this period a value for R_{ct} is calculated from the potential shift applied divided by the current induced. For greater accuracy an average value for R_{ct} is taken using both a positive and negative potential shift. The corrosion current density, i_{cor} , is determined by dividing the total corrosion current, I_{cor} , by the surface area of the steel being polarised. From the corrosion current density, i_{cor} , it is possible to calculate the annual steel section loss, which enables an estimation of the service life.

A number of uncertainties arise with the use of LPR measurements. The area of bar that is being polarised is unknown, due to an uncertain degree of lateral spread of the applied current. The development of a guard ring to limit the spread of the polarisation current has gone some way to addressing this problem (Feliu et al. 1996). Another problem is the choice of a suitable equilibration time at which to monitor the current response to the applied potential shift. This is due to the fact that R_{ct} is made up from a number of different mechanisms occurring within the concrete. These include those interfacial resistances at the surface of the bar which are directly related to corrosion and those resistances associated with the bulk concrete. To calculate the corrosion rate

accurately it is necessary to select the measurement time required to evaluate those processes directly associated with corrosion but to exclude those processes associated with the bulk concrete.

The AC impedance technique is a laboratory method which enables greater information to be derived relating to the mechanisms occurring within the system. However, this technique is particularly time consuming and can often provide data which is difficult to interpret.

A technique that has recently been developed to study the dynamic response to an applied current has been that of Galvanostatic Pulse (Newton & Sykes 1988, Gowers et al 1996, Millard et al. 1995). In this technique a small current perturbation is applied to a steel bar using an auxiliary electrode on the surface of the concrete in a similar manner to a LPR measurement. The resulting potential transient is then monitored with respect to a reference electrode. Analysis of this transient response allows the different resistance and capacitance components of the system to be resolved, enabling an accurate corrosion rate to be determined.

2 Experimental

2.1 Galvanostatic Pulse

The galvanostatic pulse experiments were conducted using a brass 15 cm x 10 cm auxiliary electrode placed on the surface of the concrete. A wetted carbon foam pad was used to ensure a good electrical contact between the auxiliary electrode and the concrete surface. This auxiliary electrode had a Silver/Silver Chloride reference electrode located in the centre, Figure 1.

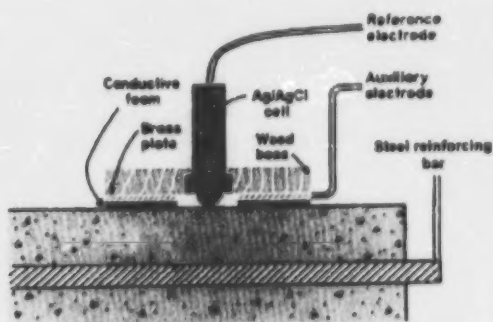


Fig 1: Galvanostatic Pulse Monitoring Equipment

A purpose built, battery powered, constant current source was used to apply the current perturbation. The current source allowed a stable 1, 0.1 or 0.01 mA current to

be applied. The resultant transient voltage response was monitored using a 16-bit data acquisition program, "DT-Vee".

A pulse duration of between 45 to 180 seconds was used with a sampling rate of 1000 Hz selected to collect the dynamic response data. This sampling rate allows those system components with small time constants to be resolved. At higher sampling rates it was observed that the resolution is not improved but the data processing times are greatly extended.

A current perturbation was selected to give an initial potential shift of 20 mV to 30 mV, which would fall within the linear region of the Stern-Geary plot. To this end initial tests were always conducted with a 0.01 mA pulse. If this did not sufficiently perturb the system the steel was allowed to re-equilibrate back to the rest potential, and the current pulse increased, as required, to achieve the desired potential shift.

2.2 Data Analysis

The data collected was analysed to obtain an equivalent electronic circuit, Figure 2, (Millard et al. 1995, Gowers et al., 1996, Gowers & Millard, in press). Using a simple Randles circuit at time t the transient potential response, V_t , to a constant current perturbation, I , is given by

$$V_t = R_\Omega \cdot I + R_{ct} \cdot I [1 - \exp(-t/R_{ct}C_{dl})] \quad (2)$$

where I is the current perturbation

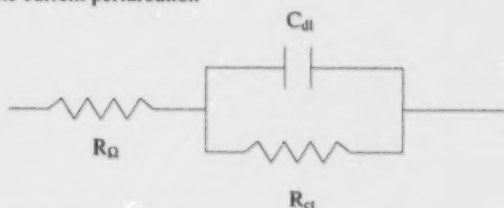


Fig. 2: Simple Randles Circuit

In this circuit R_Ω represents the ohmic resistance of the concrete between the surface electrode and the steel bar. C_{dl} represents a double layer capacitance at the surface of the steel bar and R_{ct} represents the charge transfer resistance at the surface of the steel bar. As seen in Equation 1, the rate of corrosion is proportional to R_{ct} .

After a suitable equilibration time the transient response reaches a steady-state potential V_{max} , where

$$\log_e (V_{max} - V_t) = \log_e (IR_{ct}) - t / R_{ct}C_{dl} \quad (3)$$

Thus a plot of $\log_e (V_{max} - V_t)$ against t gives a linear graph with gradient $1/R_{ct}C_{dl}$ and intercept $\log_e (IR_{ct})$. In more complex system with several separate components there will be a succession of linear sections of the graph, which can be represented by a succession of resistors and capacitors in series. By extracting each successive section

the separate components can be resolved and the R_{ct} and C_{dl} determined for each component. Knowledge of the R_{ct} and C_{dl} enables the time constant for those components to be calculated as the product of the capacitance and resistance.

2.3 Concrete Specimens

Galvanostatic pulse experiments were conducted on two concrete specimens. Both specimens were made of OPC concrete with a nominal 28 day compressive cube strength of 50 Nmm^{-2} . Specimen #1 was a slab of dimension $1000 \times 1000 \times 150 \text{ mm}$, with two mild steel bars, 10 mm diameter, 100 mm long, denoted MS1 and MS2. The bars had a cover of 25 mm, were positioned parallel to each other 50 mm apart, and located 500 mm from the edge of the slab. Specimen #2 was a slab of dimensions $750 \times 750 \times 150 \text{ mm}$, with one mild steel bar, denoted MS3, and two stainless steel, denoted SS1 and SS2. Each bar was 10 mm diameter, 100 mm long. The bars had a cover of 25 mm, were positioned parallel to each other 25 mm apart, and located 250 mm from the edge of the slab. The surface area of all the bars was taken as 31.42 cm^2 .

Each specimen was subjected to the an external environment and intermittent ponding with 1 Molar sodium chloride solution to promote corrosion over a period two years. Potential transients were recorded over time periods of between 45 and 180 seconds and at lateral distances ranging from 0 to 300 mm from the location of the bar.

3 Results

The potential transients determined had different characteristic shape for the active (mild steel) and passive (stainless steel) systems. A typical transient for an active system is given in Figure 3 and for a passive system in Figure 4. The data shows that a significantly higher potential shift, ΔE , is obtained from a passive system than an active system for the same current perturbation.

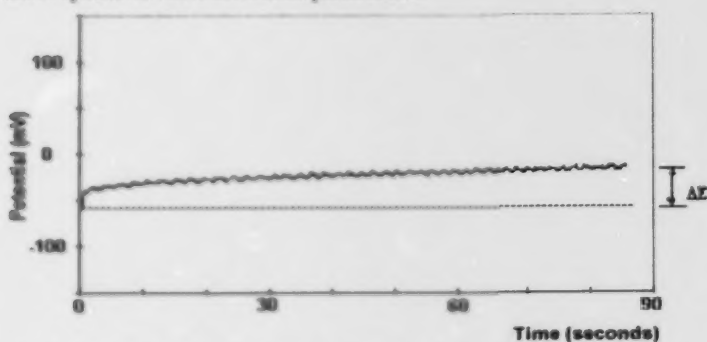


Fig. 3: Potential Transient, MS3, Active Corrosion, Pulse 0.01 mA

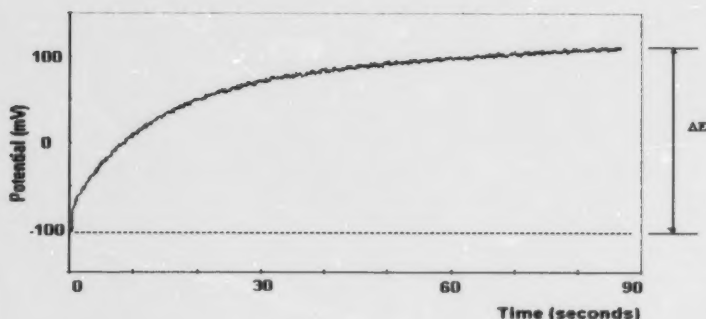


Fig. 4: Potential Transient, SS2, Passive Corrosion, Pulse 0.01 mA

For the passive systems measured the potential shift, ΔE for the lowest current perturbation, 0.01 mA, was larger than that in the linear region of the Stern-Geary plot. This may result in some inaccuracies in the resolution of the components of the transient, however, these will not be significant when converted to corrosion rates. For the active corrosion systems at higher corrosion rates a current pulse was selected to obtain the 20 - 30 mV shift required to fall within the Stern-Geary linear region. This shift was achieved in all the systems measured.

Analysis of the potential transients enabled a number of separate components with different time constants to be resolved. The resistance, capacitance and associated time constant for each component are given in Tables 2-6. The resistances in **bold** are those attributed to the charge transfer resistance.

4 Discussion

When investigating the condition of reinforcing steel in concrete the corrosion rates are generally classified as passive, low, medium or high based on the magnitude of corrosion current density i_{corr} , Table 1. In order to calculate the corrosion current density from the galvanostatic pulse data acquired, the charge transfer resistance, R_{ct} must be determined. This value can be calculated by summing the individual resistances of the separate components, associated with corrosion, resolved from the galvanostatic pulse experiments.

The analysis of the transient response from the stainless steel bars, SS1 and SS2, gave components with resistance in the range 400 to 500 $\Omega \text{ cm}^2$, which are consistent with a passive system. This is to be expected as the stainless steel bars will not be corroding. As the distance from the bar increased there was no discernible increase in the magnitude of any of the individual components. This would indicate that the components resolved are associated with corrosion at the steel/concrete interface and are not due to bulk effects in the concrete, which would be expected to increase with an increase in lateral distance. The capacitances associated with these components are all

Table 2: Results for bar MS1, 90 seconds pulse

Distance (mm)	R ₀ (Ω cm ⁻²)	R ₁ (Ω cm ⁻²)	Cd ₁ (μ F/cm ²)	t ₁ (s)	R ₂ (Ω cm ⁻²)	Cd ₂ (μ F/cm ²)	t ₂ (s)	R ₃ (Ω cm ⁻²)	Cd ₃ (μ F/cm ²)	t ₃ (s)	R ₄ (Ω cm ⁻²)	Cd ₄ (μ F/cm ²)	t ₄ (s)
0	9341	2312	11392	26.6	3280	961	3.18	4592	79	0.36			
100	27210				3749	1560	5.85	2842	301	0.86	3186	10.2	0.032
200	26117	5936	1963	11.7	3436	618	2.14				3217	20.9	0.067
300	31927	7466	3398	25.5	3842	1478	5.78				3842	35.2	0.14

Table 3: Results for bar MS2, 90 seconds pulse

Distance (mm)	R ₀ (Ω cm ⁻²)	R ₁ (Ω cm ⁻²)	Cd ₁ (μ F/cm ²)	t ₁ (s)	R ₂ (Ω cm ⁻²)	Cd ₂ (μ F/cm ²)	t ₂ (s)	R ₃ (Ω cm ⁻²)	Cd ₃ (μ F/cm ²)	t ₃ (s)
0	8497	3873	1773	6.9	4717	609	2.9	7685	13.0	0.10
100	14745	6717	1372	9.3	10215	239	2.46	8903	19.4	0.17
200	38706	8060	1424	11.5	11965	240	2.88	6623	54.1	0.36

Table 4: Results for bar SS1, 90 seconds pulse

Distance (mm)	R ₀ (Ω cm ⁻²)	R ₁ (Ω cm ⁻²)	Cd ₁ (μ F/cm ²)	t ₁ (s)	R ₂ (Ω cm ⁻²)	Cd ₂ (μ F/cm ²)	t ₂ (s)	R ₃ (Ω cm ⁻²)	Cd ₃ (μ F/cm ²)	t ₃ (s)
0	22868	375 000	64.4	24.3	178 000	40.8	7.3	45 900	2.10	0.10
50	23836	544 000	42.3	23.1	109 000	38.5	4.2	46 200	0.60	0.03
100	43861	412 000	52.5	21.8	52 500	43.0	2.3	41 900	0.31	0.01

Table 5: Results for bar SS2, 90 seconds pulse

Distance (mm)	R_0 ($\Omega \text{ cm}^{-2}$)	R_1 ($\Omega \text{ cm}^{-2}$)	Cd_1 ($\mu\text{F}/\text{cm}^2$)	t_1 (s)	R_2 ($\Omega \text{ cm}^{-2}$)	Cd_2 ($\mu\text{F}/\text{cm}^2$)	t_2 (s)	R_3 ($\Omega \text{ cm}^{-2}$)	Cd_3 ($\mu\text{F}/\text{cm}^2$)	t_3 (s)
0	13339	475 000	45.6	21.8	115 000	31.8	3.65	45 000	0.83	0.037
50	40039	540 000	38.2	20.8	91 800	40.6	3.75	45 300	1.69	0.077
100	69603	359 000	58.0	20.4	40 100	68.3	2.80	23 100	2.32	0.054

Table 6: Results for bar MS3, 45 and 90 seconds pulse

Pulse (sec)	Distance (mm)	R_0 ($\Omega \text{ cm}^{-2}$)	R_1 ($\Omega \text{ cm}^{-2}$)	Cd_1 ($\mu\text{F}/\text{cm}^2$)	t_1 (s)	R_2 ($\Omega \text{ cm}^{-2}$)	Cd_2 ($\mu\text{F}/\text{cm}^2$)	t_2 (s)	R_3 ($\Omega \text{ cm}^{-2}$)	Cd_3 ($\mu\text{F}/\text{cm}^2$)	t_3 (s)	R_4 ($\Omega \text{ cm}^{-2}$)	Cd_4 ($\mu\text{F}/\text{cm}^2$)	t_4 (s)
45	0	13339	51546	352	18.2	12464	379	4.75				11434	0.31	0.004
45	50	16214	61305	294	18.7	14908	427	6.30				22993	0.14	0.006
45	100	33364	44986	371	16.8	8622	276	3.36				4936	0.38	0.002
90	0	13714	55607	515	28.7	18213	583	5.69				5561	0.89	0.005
90	50	19150	56232	563	31.8	18994	608	6.18	5498	5.36	0.30			
90	100	21743	51858	564	29.4	18307	586	5.95	3718	24.7	0.092			

Table 7: Results for bars SS1 and SS2, 180 seconds pulse

Bar	Distance (mm)	R_0 ($\Omega \text{ cm}^{-2}$)	R_1 ($\Omega \text{ cm}^{-2}$)	Cd_1 ($\mu\text{F}/\text{cm}^2$)	t_1 (s)	R_2 ($\Omega \text{ cm}^{-2}$)	Cd_2 ($\mu\text{F}/\text{cm}^2$)	t_2 (s)
SS1	0	17150	344 000	89.2	30.9	96 800	78.0	7.6
SS	50	20025	647 000	60.5	39.4	226 000	43.3	9.8
SS1	100	39081	650 000	54.5	35.5	176 000	44.6	7.3
SS2	0	12402	338 000	77.1	26.2			
SS2	50	25742	358 000	52.4	22.4	53 100	102	5.4
SS2	100	29553	197 000	109	21.5	31 600	170	5.4

less than $100 \mu\text{F}/\text{cm}^2$ and their time constants are all less than 30 seconds and would confirm that a measurement period of at least 30 seconds should be adopted for LPR measurements. Indeed for passive systems a measurement period of 120 to 300 seconds is often employed. Additional galvanostatic pulse data acquired using a pulse duration of 180 seconds, Table 7, again showed no significant increase in resistivity with distance for the components associated with the longer time constants.

In the active systems studied there was a marked difference between the data for the specimens MS1 and MS2 and specimen MS3. Specimen MS1 displayed an increase in resistance with distance for that component with the largest time constant. Specimen MS 2 displayed an increase in resistance for the components with the two longest time constants. However specimen MS3 displayed no increase in resistance with distance. The capacitances associated with the components displaying this increase in specimens MS1 and MS2 were greater than $1000 \mu\text{F}/\text{cm}^2$, for those with time constants in the region of twenty five seconds, for bar MS2 the capacitances associated with the second time constant, 2.75 seconds, were between 609 and $239 \mu\text{F}/\text{cm}^2$. All the capacitances in specimen MS3 were less than $1000 \mu\text{F}/\text{cm}^2$.

Overall, those components in the region $100 - 1000 \mu\text{F}/\text{cm}^2$ appear to be related to either corrosion or bulk diffusion, those above $1000 \mu\text{F}/\text{cm}^2$ more likely related to bulk diffusion, while those less than $100 \mu\text{F}/\text{cm}^2$ are almost certainly related to corrosion. This is in general agreement with previous work (Newton & Sykes 1988, Gowers et al 1996, Millard et al 1995, Glass et al 1997, Glass et al 1998).

In addition there is evidence that those resistive components relating to the dielectric properties of the concrete may be resolved with time constants in the millisecond range. In the data collected components within these bounds generally display capacitances of less than $1 \mu\text{F}/\text{cm}^2$. In calculating corrosion rates from galvanostatic pulse data components associated with very small capacitances should not be included in the calculations.

A comparison of the corrosion rate calculated for all components with time constants less than 30 seconds, corresponding to a LPR measurement with a 30 second equilibrium period, and the corrosion rate calculated when the components associated with the bulk diffusion effects are discounted, is given in Table 8.

Table 8: Corrosion Rates for Mild Steel, 90 seconds pulse, Lateral distance 0 mm

Bar	ΣR_{ct} ($\Omega \text{ cm}^{-2}$)	Corrosion Rate (R_{ct}) (μAcm^{-2})	ΣR_{Total} ($\Omega \text{ cm}^{-2}$)	Corrosion Rate(R_{Total}) (μAcm^{-2})
MS1	7872	3.16	10184	2.44
MS2	7560	3.15	16276	2.42
MS3	54857	0.38	65323	0.38

The data shows that a significant underestimate of the corrosion rate may occur for those where a 30 second equilibrium period is used. Hence, the selection of an appropriate LPR equilibrium time is important when monitoring actively corroding structures.

5 Conclusions

1. The galvanostatic pulse method enables the separate components of the charge transfer resistance to be resolved. This in turn enables those components associated with the corrosion process to be identified and a corrosion rate calculated.
2. The technique can readily differentiate between active and passive systems.
3. Components with capacitances in the range $1\text{--}100\ \mu\text{F}/\text{cm}^2$ are associated with corrosion. Capacitances over $100\ \mu\text{F}/\text{cm}^2$ may be attributable to either the corrosion process or bulk diffusion effects, though capacitances greater than $1000\ \mu\text{F}/\text{cm}^2$ appear more likely to be associated with bulk effects. Each system must be individually investigated to determine which resistance components should be used to evaluate the rate of corrosion.
4. Components with capacitance less than $1\ \mu\text{F}/\text{cm}^2$ may be attributed to the dielectric properties of the concrete.
5. In structures with actively corroding reinforcing steel, the equilibrium period used for LPR measurements should be carefully selected to prevent an underestimate of the corrosion rate.

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31 THE EVALUATION OF FLEXURAL STRENGTH OF RC BEAMS DAMAGED BY REBAR CORROSION

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Abstract

This study was carried out to investigate quantitatively the relationship between the degree of rebar corrosion and the flexural strength of RC beams damaged by rebar corrosion. In the experiment, the static loading tests of RC beams were conducted at different degrees of rebar corrosion, which was controlled by an electric supply system. Also, after producing equations for the relationship between both the tensile properties of rebars and bond properties and the corrosion percentage of rebars, finite element analysis was conducted. As a result, it was made clear that the flexural strength of RC beams damaged by rebar corrosion could be practically simulated by FEM using experimentally determined material models representing the bond properties and the mechanical properties of corroded rebars.

Keywords: Rebar corrosion, RC beam, flexural strength, corrosion percentage, FEM

1 Introduction

In order to ensure safety of RC structures whose reinforcing steel has been severely corroded, it is necessary not only to repair the damage appropriately, but also to evaluate strength of RC members according to the degree of rebar corrosion. The effects of rebar corrosion on the strength of RC members reinforced with them can be evaluated by three factors; the losses in the effective cross-sectional area of concrete due to cracking of cover concrete; the losses in the mechanical properties of rebars due to their reduced cross-sectional areas; and the bond strength and rigidity between corroded rebars and concrete. Also, it was confirmed that the strength of RC

material models representing the relationships between the degree of rebar corrosion and the bond properties and mechanical properties of corroded rebars can be derived. The purpose of this study is to investigate quantitatively and experimentally the relationship between the degree of rebar corrosion and the flexural strength of RC beams damaged by rebar corrosion. Also, results of experiment on RC beams damaged by rebar corrosion were compared with the results of finite element analysis.

2 Flexural loading tests of RC beams damaged by rebar corrosion

2.1 Experimental procedure

Table 1 shows the types of specimen. In this experiment, three types of specimens were used: sound specimen (BS); model specimen with tension rebars of reduced size (BD); and corroded specimens which are identical to BS specimen except that their main tension rebars have been corroded in advance by an electrolytic corrosion method (BCD1-0 corrosion step 1, BCD2-0 corrosion step 2, and BCD3-0 corrosion step 3). The details of RC beams is shown in Fig. 1. The specimens were 200x250x2400 mm in size and the shear span ratio is 3.4.

Table 1: Types of specimen

No.	Symbol	Names of specimen	Tension Main Rebar	Common Data
1	BS	Sound	3-D13 ($P_r=0.87\%$)	Cross section
2	BD	Model	3-D10 ($P_r=0.49\%$)	$b \times D = 200 \times 250 \text{ mm}$
3	BCD1-0	Corrosion 1	3-D13 ($P_r=0.87\%$)	Shear Span Ratio: 3.4
4	BCD2-0	Corrosion 2	3-D13 ($P_r=0.87\%$)	Stirrup ratio
5	BCD3-0	Corrosion 3	3-D13 ($P_r=0.87\%$)	D6-@50 (0.64%)

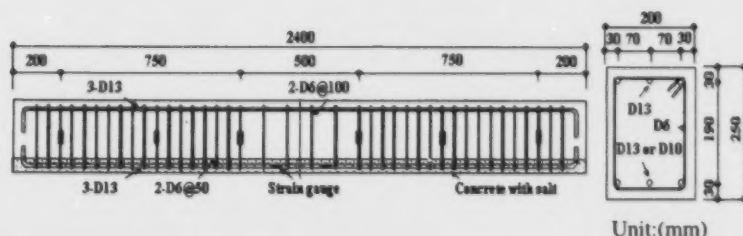


Fig. 1: Details of RC beams

2.2 Materials properties

The mixing proportion and mechanical properties of concrete are shown in table 2. Also, the mechanical properties of rebars are shown in Table 3.

Table 2: Mixing proportion and mechanical properties of concrete

W/C (%)	s/a (%)	Slump (cm)	Unit Weight (kg/m ³)				σ_c (N/mm ²)	E_c (N/mm ²)	v
			W	C	S	G			
65	45	18	185	285	776	1007	39.2	29419	0.19

Table 3: Mechanical properties of rebars

Type	Yield Strength	Tensile Strength	Elastic Modulus
	(N/mm ²)	(N/mm ²)	(Gpa)
D13	343	477	182
D10	289	429	187
D 6	226	415	194

2.3 Corrosion methods of tension main rebars for RC beams

The electrolytic corrosion method is shown in Fig. 2. The level of corrosion was controlled by impressing a direct current of 1A on the specimens for specified periods using a constant electrical current supply. The current was so arranged that the rebar embedded in the specimens and a copper plate, both immersed in water, served as the anode and cathode, respectively. The corrosion percentage was determined by measuring the weight of the rebars after removing the rust with 10% diammonium hydrogen citrate.

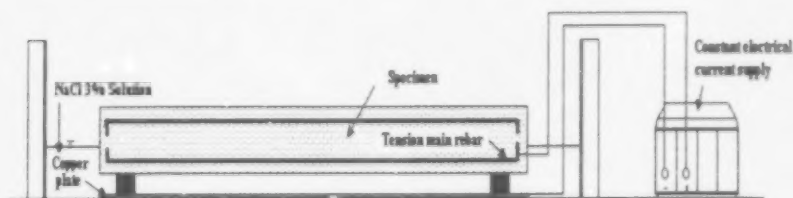


Fig. 2: Electrolytic corrosion method

2.4 Static loading tests and measurement items

In loading the specimen, concentrated loads were applied unidirectionally and monotonically at two points, as shown in Fig. 3. In the tests including sound specimen BS, model specimen BD and corroded specimen BCD1-0, BCD2-0 and BCD3-0, beam deflections at the loading points, strains in the tension main rebar and crack width were measured, and the progress of crack and the condition of crushing failure of concrete were recorded.

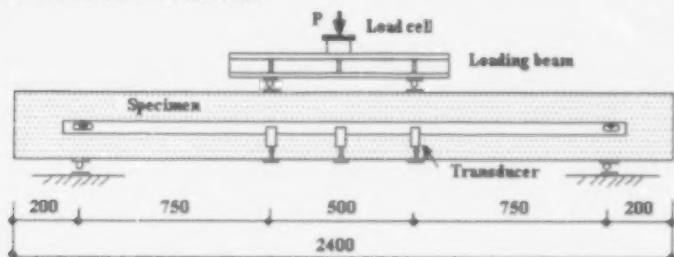


Fig. 3: Static loading test

2.5 Results and discussion

Table 4 is a list of the test results. Fig. 4 gives an overall view of cracking in the specimens observed after carrying out electrolytic corrosion. Mainly cracks alongside the main tension rebar occurred and the corrosion crack width increased with the corrosion level. Also, the rebars had black rust caused by the electrolytic corrosion all over and their surfaces had flake rust and partial losses of the sectional area. Fig. 5 shows cracks observed in the specimens after they failed. The BS and BD specimens collapsed by crushing of concrete after many flexural cracks occurred. The corroded specimens BCD1-0, BCD2-0 and BCD3-0 showed a concentration of flexural cracks in the even bending section. The spalling of concrete at the bottom of the specimen indicated that the bond between tension main rebars and concrete had deteriorated. The load-deflection relationships for specimens are shown in Fig. 6. The maximum strength of BD was 70% that of BS, and the rate of decrease in strength was small for the amount of reduction in the cross-sectional area of rebars. Also, as the corrosion of reinforcing bars progressed, the yield strength and the maximum strength decreased, along with deflection at the maximum strength. Fig. 7 shows the relationship between corrosion percentage (Δw) and the maximum strength ratio (strength of BS / strength of corrosion specimens). As the corrosion percentage increased, the yield strength and maximum strength of the corrosion specimens decreased due to the reduction of sectional area of corrosion rebars, the bond failure of corrosion rebars and the falling of cover concrete.

Table 4: Test results

Symbol	Δw (%)	Cracking Load (kN)	Yield Load (kN)	Maximum Load (kN)	Type of Failure
BS	0	22.6 (1.00)	75.5 (1.00)	88.9 (1.00)	Flexure
BD	0	25.5 (1.13)	49.0 (0.65)	62.6 (0.70)	Flexure
BCD1-0	3.8	21.6 (0.96)	71.1 (0.94)	85.4 (0.96)	Bond and flexure
BCD2-0	7.9	20.6 (0.91)	69.6 (0.92)	78.8 (0.89)	Bond of rebar
BCD3-0	25.3	24.5 (1.09)	51.5 (0.68)	67.3 (0.76)	Bond of rebar

Δw : corrosion percentage by weight, (): Strength of specimen / Strength of BS

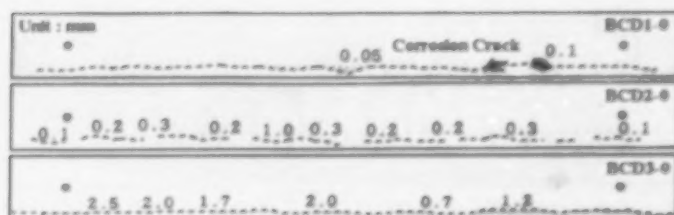


Fig. 4: Cracks in concrete cover due to rebar corrosion

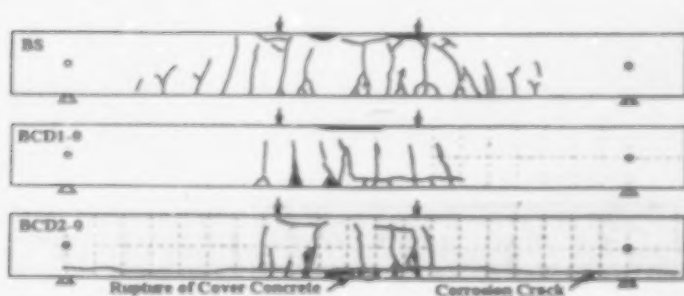


Fig. 5: Cracks patterns of RC beams at failure

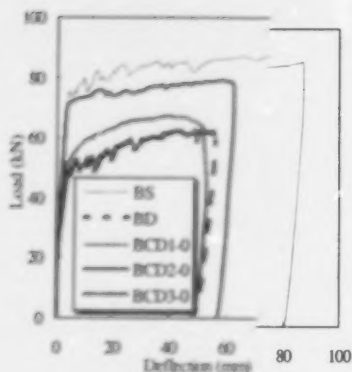
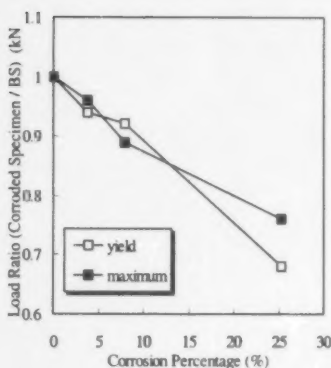


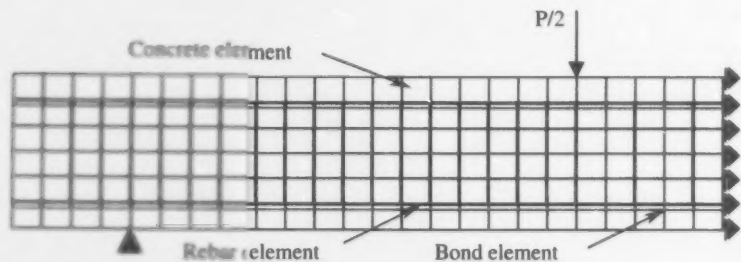
Fig. 6: Load - deflection curves

Fig. 7: Relationship between Δw and strength of RC beams

3 FEM analysis of RC beams damaged by rebar corrosion

3.1 Outline of FEM analysis

The specimens analyzed by the finite element method were the sound specimen BS, the model specimen BD and corrosion specimen BCD1-0, BCD2-0 and BCD3-0. Because of the symmetry of the specimens, only the left half of each specimen was analyzed. Each specimen was modeled by dividing the specimen into elements as shown in Fig. 8. The analysis program is a two-dimensional program, which is a type of non-linear analysis program. The displacement-increment method were used for loading.



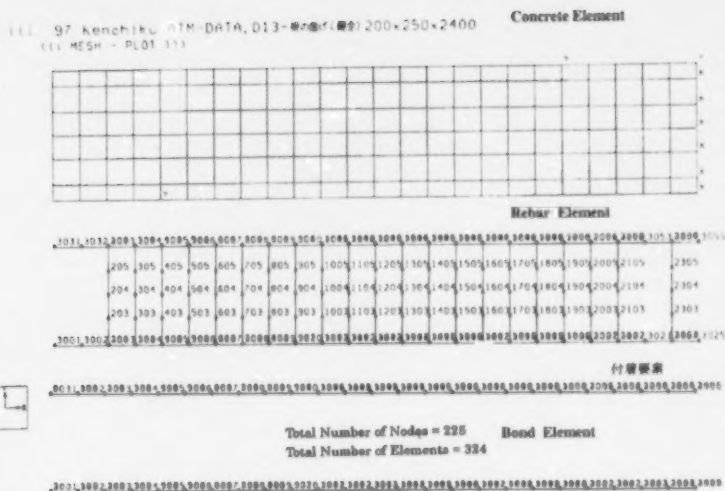


Fig. 8: Finite element meshes for FEM analysis

3.2 Constitutive laws for material elements

- 1) Concrete elements: Being an isotropic material, 4-node isoparametric plane stress elements were applied. The main input data were compressive strengths, Poisson's ratios and tensile strengths obtained from the experiment.
- 2) Rebar elements: The bilinear model shown in Fig. 9 is a model available for tension main rebars in an FEM analysis. The main tension rebars and stirrups were expressed by truss elements and the main input data were yield points and elastic modulus of rebars. The mechanical properties of corroded rebars were calculated using the constitutive law based on electrolytic corrosion in Table 5. Accordingly, the constitutive law for rebars can be obtained by determining the yield point and elastic modulus of rebars using the degree of rebar corrosion as a parameter (Δw).
- 3) Bond elements between rebars and concrete: Bond elements consist of the maximum bond strength (τ_{max}) and bond stiffness (D_s) as shown in Fig. 10. Accordingly, the constitutive laws for bond elements can be obtained by determining τ_{max} and D_s using the degree of rebar corrosion as a parameter. In the FEM analysis, bond elements were expressed by changing the τ_{max} and D_s . Also, 4-node isoparametric plane bond elements used as the bond elements. Their physical properties were calculated from the corrosion percentage using the equations in Table 6.

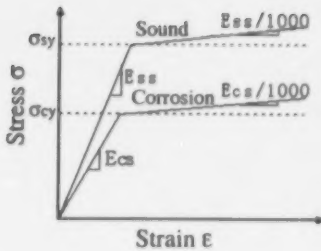


Fig. 9: Modelling of corrosion rebar

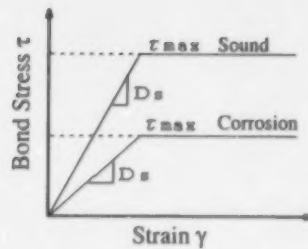


Fig. 10: Modelling of bond

Table 5: Equations for mechanical properties of corroded rebar

Properties	Type of Corrosion	Equations
Yield*	Uniform Corrosion	$\sigma_{cy} = (1-1.24(\Delta w/100)) \times \sigma_{sy}$
Strength	Pitting	$\sigma_{cy} = (1-1.98(\Delta w/100)) \times \sigma_{sy}$
Elastic*	Uniform Corrosion	$E_{cs} = (1-1.24(\Delta w/100)) \times E_s$
Modulus	Pitting	$E_{cs} = (1-1.24(\Delta w/100)) \times E_s$

* Nominal values, σ_{sy} : sound rebar, σ_{cy} : corroded rebar

Table 6: Equations for bond properties of corroded rebar

Corrosion percentage (%)	Bond strength (τ_{max}) (Mpa)	Bond rigidity (D_s) (Mpa)
$\Delta w < \Delta w_c$	$\tau_{max} = 0.34\sigma_B - 1.93$	$D_s = 16.5\sigma_B - 160$
$\Delta w \geq \Delta w_c$	$\tau_{max} = 5.21e^{-0.0561\Delta w}$	$D_s = 11600\Delta w^{-1.014}$

σ_B : Strength of concrete, Δw : Corrosion percentage Δw_c : Δw at cracking

3.3 Results of FEM analysis

Fig. 10 shows the measured and analyzed state of cracking after failure. In sound specimen (BS), bending cracks spread over to the shear zone, exhibiting a bending failure mode due to collapse of compressed concrete. In corroded specimen (BCD3-0), however, cracks tended to concentrate on the equal bending moment zone, exhibiting a bonding failure mode associated with widening of axial corrosion cracking due to corrosion expansion of rebars and spalling of cover concrete. Though the FEM analysis results agreed well with the experiment results, a number of cracks appeared along the rebars on the sound beam, while few cracks appeared along the rebars on the beam with corroded rebars. This may be because the bond failure between the rebar and concrete due to rebar corrosion in effect prevented the stress on the rebars from being transferred to concrete.

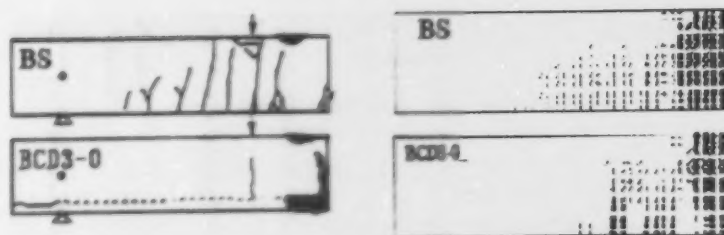


Fig. 10: Crack patterns of RC beams (FEM analysis)

Fig. 11 shows the load-deflection relationships obtained from the loading tests on the reinforced concrete beam and the FEM analysis of sound beam BS and model specimen BD. The analyzed load-deflection relationship agreed well with the static loading test results. Therefore, it can be said that the validity of the input data and the FEM analysis verified. Fig. 12 shows the load-deflection relationships obtained from the loading tests on the reinforced concrete beam and the FEM analysis of corroded beams BCD2-0 and BCD3-0. As a result, the FEM analysis results agreed well with the static loading test results. Accordingly, it is considered that the losses in the structure performance of concrete beams with corroded rebars can be evaluated by an FEM analysis by evaluating the physical properties of rebar elements and bond elements based on the corrosion percentage by weight.

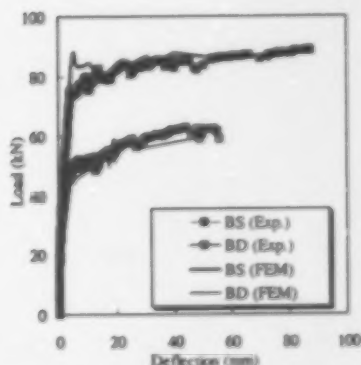


Fig. 11: Load - deflection curves

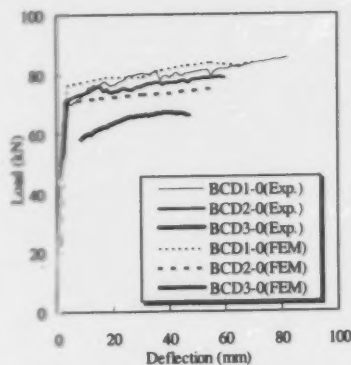


Fig. 12: Load - deflection curves

4 Conclusions

To investigate quantitatively the relationship between the degree of rebar corrosion and the flexural strength of RC beams damaged by rebar corrosion, the static loading test and FEM analysis of RC beams were conducted. The results obtained from these experiments and analysis are summarized as follows :

- 1) If the main tension rebars of RC beam were corroded, mainly cracks alongside the main tension rebar occurred. Also, the corrosion crack width increased with the corrosion level.
- 2) As the corrosion of main tension rebars progressed, the yield strength and the maximum strength of RC beams decreased, along with deflection at the maximum strength due to the reduction of sectional area of corrosion rebars, the bond failure of corrosion rebars and the falling of cover concrete.
- 3) The FEM analysis results of RC beams damaged by tension main rebar corrosion agreed well with the static loading tests results of that. Accordingly, it is considered that the losses in the structure performance of concrete beams with corroded rebars can be evaluated by an FEM analysis, if the mechanical properties of rebar elements and bond elements based on the corrosion percentage by weight can be derived.
- 4) The subjects to be investigated in the future include the accuracy improvement of the constitutive law for each material changed by corrosion of rebar. Also, the FEM analysis results should be verified with full-scale loading tests not only on RC beams but also on RC column and RC slab.

5 Acknowledgements

The study was supported by Grant-in -Aid for Developmental Scientific Research (A), The Ministry of Education, Science and Culture (Subject No. 06402045), to which the authors are grateful. Also, the authors would like to thank Kanetoh Masayoshi, Total Information Service Company, for his help in the FEM analysis and Building Research Institute of Japan.

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32 STEEL CORROSION IN CONCRETE: A COMPREHENSIVE EXPERIMENTAL PROGRAM AND PRELIMINARY RESULTS

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Abstract

The emphasis of current research in steel corrosion is on the corrosion mechanisms and on the prevention and mitigation of corrosion from a material perspective. Little is known about how the material induced degradation affects the residual strength and serviceability of a concrete structure in a quantitative way. The intention of this paper is to discuss issues related to steel corrosion in concrete structures. A comprehensive experimental program aiming to investigate strength and serviceability deterioration of concrete structures is presented. The thrust of the program is that full size specimens are tested under service loading and subjected to simulated environment in an environmental chamber. The program will provide vital data on corrosion initiation, propagation and its effect on residual strength and serviceability of concrete buildings which can be used as a rational basis for rehabilitation and replacement of deteriorated concrete structures. Some preliminary results are also presented.

Keywords: Reinforcing steel, corrosion, concrete structures, residual strength

1 Introduction

Reinforced concrete structures are one of the most widely used engineered structures and are frequently subjected to extreme natural hazards, such as wind, earthquake etc. These structures also suffer material and structural deterioration to certain extent. Of three major mechanisms that cause structural concrete deterioration (Warner, et al 1989), steel corrosion in concrete is the most severe form of deterioration and is most difficult to deal with.

Considerable research on reinforcing steel corrosion has been undertaken. But the emphasis of current research is on the corrosion mechanisms and on the prevention and mitigation of corrosion from a material perspective. Little is

known about how the material induced degradation affects the residual strength and serviceability of in-service concrete structures in a quantitative way. The corrosion induced structural deterioration is attributed to a combination of factors including area reduction and bond loss. This combined effect of steel corrosion on structural performance has not been fully investigated and an understanding of this effect can only be gained through experiment. Prediction of the remaining safe life of corrosion-affected structures must be based on models of structural deterioration (strength and serviceability). Such models can only be derived from realistic and accurate data. Field data is usually poor in accuracy and has a high degree of variation (uncertainty). Laboratory data so far is obtained from tests without a service load and hence they are not fully applicable to structures in-service (i.e. under loading). Research into the corrosion induced bond loss and its effect on structural strength and serviceability under operating loads is virtually non-existent as shown in literature, due mainly to a lack of experimental facilities capable of simulating a desired aggressive environment and maintaining load during the corrosion process.

The intention of this paper is to discuss issues related to steel corrosion in concrete structures. Based on the understanding of the problem, a comprehensive experimental program aiming to investigate strength and serviceability deterioration of concrete structures, caused by corrosion of reinforcing steel in concrete, is presented. The thrust of the program is that service loading is to be applied to test specimens which at the same time are subjected to simulated marine environment in a large corrosive environmental chamber with salt spray facility. The proposed research will provide vital data on physical characteristics of corrosion initiation, propagation and its effect on residual strength and serviceability of concrete structures which can be used as a rational basis for rehabilitation and replacement of deteriorated concrete structures. Some preliminary results are also presented.

2 Current issues related to steel corrosion

It is believed that research into structural durability should be focused on how to assess the performance over the time during the service life of the structures. Only in this way can the research of structural durability be of any practical significance and the benefit to the society and community be observed. Since the factors that affect structural durability are highly uncertain and time-variant, only a time-dependent reliability approach is justifiable in dealing with the problem. The theoretical framework for structural assessment based on time-dependent reliability theory has been laid but substantial work is needed, particularly in experimental research, to make the methodology practically useable. Based on this understanding it can be envisaged that the followings are demanding issues in the research of concrete structural durability.

2.1 Initiation of steel corrosion under service loads

A theoretical prediction of initiation time of steel corrosion in concrete is through Fick's law to estimate the diffusion of Cl^- to the surface of the steel. Unfortunately, the assumptions built in this law in application to initiation prediction are not always met in practice. In particular, the service load induced

cracks would invalidate the assumption that concrete be homogeneous material, even though it may be assumed to be homogeneous in the absence of cracks. Also the diffusion process assumed in Fick's law applies to "semi-infinite" solids with constant concentration of chloride on the surface, which limits its application to more common situations, in particular, the chloride concentration on the surface varies. Also half cell potential measurement for the initiation of steel corrosion seems to be very random. It is therefore desirable that a model be developed to predict the initiation of steel corrosion in a situation that is more realistic, i.e., a concrete structure in service under loading and aggressive environment.

2.2 Residual strength

As has been noted steel corrosion is essentially a two-stage electrochemical oxidation process. The current research should be focused on the second stage, i.e., determining the strength deterioration which depends on corrosion rate and, in turn, various factors that are related to material and structural properties. However, very limited research has been undertaken using advanced stochastic approaches. Although in some countries, e.g., in Europe, considerable research has been undertaken (Andrade, et al 1996), care should be taken in considering the fact that substantial differences exist (in different countries) in environments, material constituents, construction practices and design loads.

Theoretical models based on area reduction of the cross-section of rebars are more or less simplistic. The model is of the form as follows (Li and Melchers 1993)

$$A_r(t) = \frac{A_r(0) - A_r(t)}{A_r(0)} \quad (1)$$

where $A_r(t)$ is the area of reinforcing steel. It is evident that the residual strength of a concrete structure is not only related to area of rebar (although it may be dominant). It depends on a combination of factors, such as the area reduction, the bond loss at the interface of steel and concrete, expansion of cracks (deepening) and so on. These factors may also interact, resulting adverse affect on the strength of the structural section. All this means that any theoretical model needs to be verified by experiment.

2.3 Acceptable probability of deterioration failure

In performance assessment, the probability of deterioration failure is checked against an acceptable probability of structural failure. How to determine the acceptable probability of structural failure is of great significance in decision-making in infrastructure (asset) management. The acceptable limit for the probability of structural failure can be determined in the framework of reliability-based economic optimisation of structures. The basic idea of economic optimisation is to find an optimal balance between the total cost of a structure (including initial cost, maintenance cost, rehabilitation cost etc.) and the safety level that is required for the structure, which should be based on a socio-economic criterion (Li 1995). Although the concept of reliability-based optimisation as illustrated in Fig. 1 is not new, the topic has not been fully explored and a general solution to obtain the optimised safety index (acceptable failure probability) and to incorporate it in engineering structures is highly desirable.



Fig. 1: Optimised safety level

The total cost, C_T , of a concrete structure (including initial cost, C_I , maintenance and/or repair cost, $C_{M,R}$, the cost for structural failure, C_F , etc.), can be expressed as

$$C_T = C_I + C_{M,R} + \dots + p_f C_F \quad (2)$$

Theoretically, a "target" acceptable limit for the failure probability can be determined by minimising the total cost of equation (2).

2.4 Durability design

Traditionally the durability design of concrete structures is based on implicit rules for materials, material compositions, working conditions, structural dimensions, etc (SAA 1994). These "deemed-to-comply" rules are also related to the type of environmental exposure. Modern structural design codes should increasingly be based on the performance of structures. It must be ensured that the required performance exists throughout the service life of the structures. With deemed-to-comply rules it is not possible to give an explicit relationship between the performance and service life. For concrete structures these relationships are not yet available as design tools. They need to be developed.

3 A comprehensive research program on steel corrosion

A comprehensive experimental program aiming to investigate strength and serviceability deterioration of concrete structures caused by corrosion of reinforcing steel in concrete has been developed at Monash University since 1997. The first step was to apply for funding to build large corrosive chamber with salt spray facility. The overall objective of the program is to develop a reliability-based methodology to quantitatively evaluate structural deterioration of concrete structures and to predict the remaining service life. The experiments proposed examine those factors from materials, service loads and exposed environment that contribute to steel corrosion, including corrosion rate and debonding, which leads to loss of structural integrity and serviceability.

3.1 Large corrosive environmental chamber

Funded by Australian Research Council, a large corrosive environmental chamber has been built in the Department of Civil Engineering, Monash University, Australia. It is believed to be a world first and largest environmental chamber with salt spray facility. The chamber has the following functions:

- ◆ Operating temperature range of 10 to 60 °C
- ◆ Operating relative humidity (RH) range of 50% to 100%
- ◆ Content of carbon dioxide of up to 4%
- ◆ Controlled salt spray facility (different content of sodium chloride, NaCl)

The features of chamber are as follows:

- ◆ Plan dimension of 4 by 8 meters
- ◆ World largest with salt spray facility
- ◆ Fully computerised
- ◆ Full size structural members can be tested
- ◆ Service loads can be applied
- ◆ Tests can be controlled and accelerated
- ◆ Different environments can be simulated

The tests that can be done include:

- ◆ Effect of material degradation on structures
- ◆ Effect of material constituents on material degradation
- ◆ Structural residual strength and deterioration rate under service loads
- ◆ Structural failure mechanisms
- ◆ Effect of material degradation on fatigue
- ◆ Diagnose and remedial strategies

3.2 Initiation of steel corrosion under service loads

This is the first project of the comprehensive program, the aim of which is to determine the initiation time of steel corrosion in concrete from a probabilistic perspective. Specifically the project examines three factors that are deemed to contribute to the initiation of steel corrosion under service conditions. These factors are service loading, chloride content in the environment and concrete constituents. The objective of the project is to develop a stochastic model to predict initiation of steel corrosion. The outcomes of the project will be:

- ◆ A quantitative understanding of the effect of service loading, chloride content and concrete constituents on initiation of steel corrosion in concrete;
- ◆ Stochastic models to predict the time of initiation of steel corrosion.

3.2.1 Details of specimens

There are two types of test specimens: full size beams and model beams. The cross-section of the specimens is as shown in Fig. 2. There are 60 model beams: 54 to be tested under accelerated conditions, including: 3 cement type; 2 water cement ratios; 3 time points to obtain a function over time; and 3 replicates for statistical study. Concrete ingredients of model beams are as shown in Table 1. There are six full size beams to be tested under accelerated conditions, consisting of different mixes of concrete with 3 cement types: ordinary Portland cement; fly ash blended cement (3:7); and blast furnace slag blended cement (3:7); and 2 water cement ratios: 0.45 and 0.6.

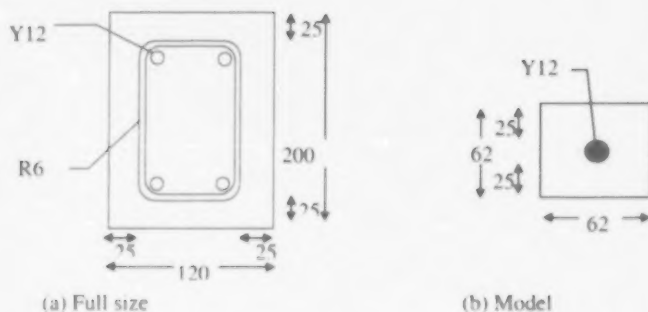


Fig. 2: Cross-section of test specimens

Table 1: Concrete mixes used in model beams

w/c = 0.45			
Ingredient	Normal Portland	Fly ash	Furnace slag
Sand	62.5 kg	62.5 kg	62.5 kg
Cement	31.4 kg	21.9 kg cement 9.6 kg fly ash	21.9 kg cement 9.6 kg slag
Coarse Aggregate	94 kg	94 kg	94 kg
Water	14.2 kg	14.2 kg	14.2 kg

3.2.2 Test method

The test rig for model beams is as shown in Fig. 3. Loads up to 60% of the ultimate flexure strength are applied to model beams by tying rods. This load is kept constant during the test duration. The acceleration is achieved by manually spraying the salt solution on the beams for wetting, and then being left in oven at 50°C for drying. There are two wet-dry cycles in a week.

The test rig for full size beams is as shown in Fig. 4, where the loads are applied using lead ingots, which are up to 60% of ultimate strength of the beam. For the full size beams the cycles will be controlled in the environmental chamber. Extra model beams and full size beams are subjected to natural cycles of wet and dry to obtain the actual cycle period of wet and dry under natural conditions.

3.2.3 Parameters to measure

Three parameters are measured in the tests: (1) half cell potential; (2) depth of chloride penetration and; (3) chloride concentration. All these parameters are measured over time. The initiation of corrosion is determined when the chloride concentration on the surface of steel bars exceeds a threshold of 0.06% of concrete by weight and this initiation is confirmed with visual inspection by breaking open the specimens after cleaning. Half cell potential is also used as an indication of corrosion.

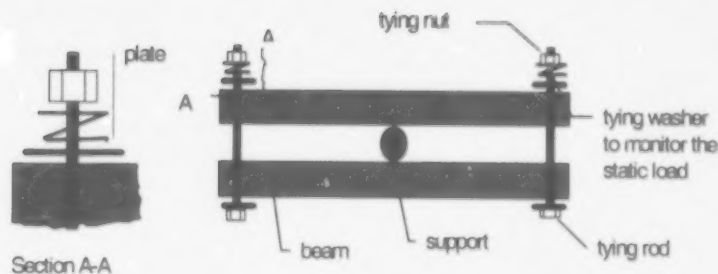


Fig. 3: Test rig for model beam specimens



Fig. 4: Sketch of test rig for full size beams (symmetric)

3.3 Determination of residual strength

The aim of this project is to investigate strength deterioration of concrete structures. In particular, it is to examine the strength deterioration due to a combination of area reduction of steel bars and bond loss. The factors to be examined are the same as in Section 3.2. The objective of the project is to develop a reliability-based methodology to quantitatively determine residual strength of concrete structures and to predict the remaining service life. The outcomes of the project will be:

- A quantitative understanding of the effect of the build-up of corrosion products on residual strength and failure mechanisms induced by corrosion;
- A stochastic model of strength deterioration of concrete structures under corrosion attack and service loadings;
- A reliability-based method to predict the remaining service life of deteriorated concrete structures.

3.3.1 Details of specimens

There are 28 beam specimens to be tested in the project, broken down as follows. 2 cement types: normal Portland cement and fly ash blended cement (3:7); 2 water/Cement ratios: 0.45 and 0.6; 2 duplicates for statistical study; and 3 time periods: 3, 6 and 9 months duration respectively. An additional 4 beams (2 cement types, 2 water/cement ratios) will be used for long term testing to verify the data acquired under accelerated conditions. Typical cross-section of the beam is as shown in Fig. 5.

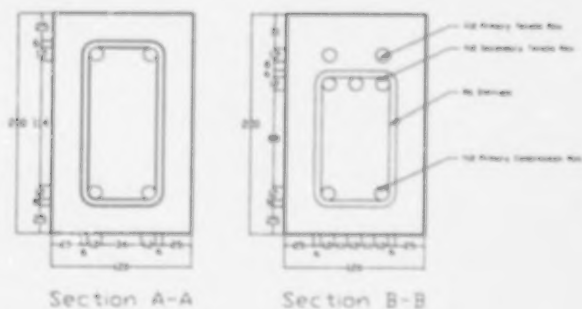


Fig. 5: Details of cross-section of fuss size beams

3.3.2. Test method

The test rig is as shown in Fig. 4. The specimens are exposed in the chamber for acceleration. A typical cycle of wet and dry is as follows: sprayed with salt water for 2 hours until saturated. The salt (NaCl) content is 3.5% by weight to comply with Australian Sea Water; dry purge at 50°C for 24 hours. Generally there are 2 cycles a week. Service loads of 60% ultimate load are applied by lead ingots and kept constant during the test duration.

3.3.2. Measurement

The following parameters will be investigated in the experiment: (1) Bond loss, which is determined by slips at the interface of concrete and steel and recorded with DeMag gauge; (2) Corrosion rate, which is determined by the cross section reduction of steel bars; (3) Ultimate strength, which is determined by external loads at cantilevered ends; (4) Deflection, which is measured at the cantilever ends; and (5) Cracks. Crack width, depth and patterns will be measured and monitored.

4 Preliminary results

Preliminary results can be summarised as follows:

- ◆ Corrosion occurs where cracks are (see Fig. 6). So Fick's law is not accurate to predict the initiation time of steel corrosion when service loads may induce considerable cracks. In the tests cracks width varied from 0.2 to 0.5 mm.
- ◆ Half cell potential measurement is very unreliable (Fig. 7). At the time of corrosion, the range of half cell potential varied from -350mV to -600 mV.
- ◆ Chloride content seems to be more accurate than other measurements (Fig. 8).
- ◆ The bond failure in a pioneer test to determined the bond strength is in Fig. 9.

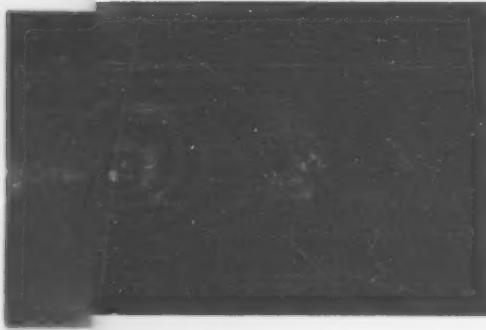


Fig. 6: Corrosion initiation at cracks

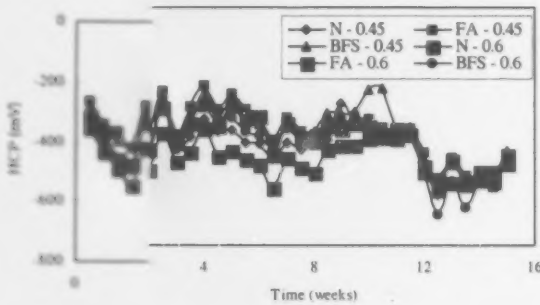


Fig. 7: Half cell potential varies with time

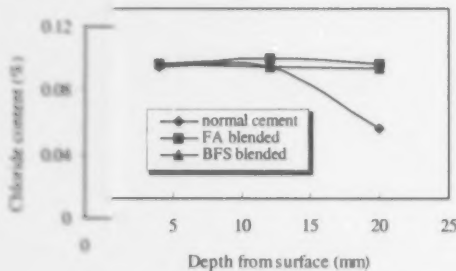


Fig. 8: Chloride content (by weight of concrete) varying with the depth from surface

- The relationship between slip and ultimate load will be presented in the length limit of the paper.
- The relationship between slip and ultimate load will be presented in the length limit of the paper.



Fig. 9: Slag Blended Cement - 200mm Transmission Length (Beam 3)

5 Conclusion

Current issues related to steel corrosion in concrete structures have been discussed. A comprehensive experimental program aiming to investigate strength and serviceability deterioration of concrete structures has been presented. Preliminary results from this program have also presented. It can be concluded that corrosion initiation and propagation in concrete structures without loading are different to those under loading. The focus of research on concrete durability should be on how to assess the structural performance and hence to determine the maintenance time and service life of the structures.

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33 COMPARISON OF CHLORIDE DIFFUSION COEFFICIENT TESTS FOR CONCRETE

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Abstract:

Corrosion service life predictions based on Fick's second law require an estimate of the chloride diffusion coefficient for concrete. Since the industry lacks a standard diffusion coefficient test procedure, several procedures are currently being used. Not surprisingly, the value of the diffusion coefficient depends on the test method employed. This paper presents the results of a three-year study aimed at producing a valid laboratory procedure to estimate the chloride diffusion coefficient for concrete. The investigation involved obtaining chloride diffusion coefficients using different laboratory procedures: natural chloride flux, chloride migration and chloride ponding. The paper discusses each test method, calculation procedure, and practical limitations. It is shown that the natural chloride flux test is the most fundamentally sound procedure to obtain the chloride diffusion coefficient, but it is too slow for practical application. The data indicate that diffusion coefficients obtained by the chloride flux and accelerated migration test methods are significantly different from chloride ponding tests. In addition, the influence of silica fume and an organic corrosion-inhibiting admixture on chloride diffusion coefficients are discussed.

Keywords: chloride flux, chloride migration, chloride ponding, chloride diffusion, chloride diffusion coefficient, service life, concrete, corrosion, Fick's second law of diffusion, organic corrosion-inhibiting admixture.

1 Introduction

Most corrosion service life models are based on Fick's second law of diffusion. In order to use these models, estimates of the chloride loading and chloride diffusion coefficient must be made. (Amey et al. 1998; Bamforth 1994; Berke and Hicks 1993; Weyers et al. 1993) Historically, these parameter estimates came from determining the chloride content at multiple depths in existing concrete structures or chloride ponding experiments. The chloride content was then mathematically fitted to Fick's second law as a function of sample depth in order to obtain the surface concentration and diffusion coefficient.

The primary drawbacks of determining the parameter estimates from fitting chloride profiles are: 1) making the incorrect assumption that diffusion is the only mechanism for chloride ingress in concrete, and 2) obtaining a chloride analysis from structures is expensive. In reality, this process requires too much time and effort to be useful for modern designers; especially those interested in evaluating new materials. In the last decade, accelerated methods to estimate the diffusion coefficient have developed (Andrade 1993; Streicher and Alexander 1995; Zhang and Gjorv 1994), but the values obtained from the different methods do not necessarily agree.

Most chloride diffusion coefficient test methods fall into three basic categories:

1. Chloride flux procedures determine the diffusion coefficient by measuring the flow of chloride through a known area of concrete over time (flux). The driving force in this test is simply a concentration gradient across the sample. The test schematic is shown in Fig.1. The diffusion coefficient is calculated from the rate of change in the chloride concentration in the sodium hydroxide reservoir.
2. Chloride migration procedures reduce the test duration by driving the chloride through concrete with an electrical potential. The diffusion coefficient calculation varies between the two migration methods typically used; one method is based on the conductivity of chloride-saturated concrete, and the other is similar to the chloride flux test with corrections for the electrical current.
3. Chloride ponding tests also have two versions; one is known as static ponding, and the other is known as bulk diffusion. Increasing the solution concentration, the ambient temperature, or both accelerates these procedures. In one procedure, a concrete specimen is ponded with a chloride solution, while in the other method the specimen is immersed in the chloride solution. After a certain time period, the chloride content data are mathematically fit to Fick's second law of diffusion as a function of depth to obtain the apparent surface concentration and diffusion coefficient.

2 Research significance

The data presented in this paper show the relationship between three chloride diffusion coefficient tests for concrete. In addition, the data show the effect of silica fume and an organic corrosion-inhibiting admixture¹ (OCI) on the chloride diffusion coefficient estimate.

3 Experimental

The data were collected over three years as part of six different evaluations. As such, data presentation has been abbreviated. Each evaluation was performed under controlled laboratory conditions following standard procedures for mixing, casting, and curing by ACI-certified technicians. All concrete mixtures used materials in conformance with ASTM specifications, and were moist-cured for 28 days prior to testing.

Two sets of specimens were exposed at Treat Island, ME, a severe weathering exposure site run by the US Army Corps of Engineers. The exposure site is a wood platform set at mean sea level, so the concrete is exposed to tidal oscillations. This facility exposes concrete to two wet/dry, or freeze/thaw cycles per day.

3.1 Mixture proportions

Since the data presented in Table 1 come from multiple evaluations, the concrete mixture proportions differ. In order to identify the pertinent experimental variables without listing individual mixture proportions, an observation number and mixture code were assigned to each mixture. The mixture code format is:

$$(c+m) - w/(c+m) - SF - OCI$$

Where:

$(c+m)$ = cementitious materials content, kg/m^3

$w/(c+m)$ = $100 \times$ water-to-cementitious materials ratio

SF = silica fume percentage by mass of total cementitious materials,

OCI = dose of organic corrosion-inhibiting admixture, L/m^3

For example, the code 356-45-6-5 indicates that the concrete contained a total cementitious content of 356 kg/m^3 ; the water-to-cementitious materials ratio was 0.45, 6 % of the cementitious materials mass was silica fume; and 5 L/m^3 of an organic corrosion-inhibiting admixture¹ were added.

¹ Rheocrete® 222+; an aqueous solution of amines and fatty acid esters, is manufactured by Master Builders, Inc., Cleveland, Ohio

3.2 Test methods

Fig. 1 presents a schematic of the test setups used to determine the chloride diffusion coefficients.

3.2.1 Chloride flux

The chloride flux procedure uses a pre-saturated 25-mm thick by 100-mm diameter concrete sample cut from a core or cast cylinder. The concrete specimen is placed in the cell so the top (finished) surface faces the sodium chloride (NaCl) reservoir, and the cut surface faces the sodium hydroxide (NaOH) reservoir. The minimum specimen thickness is twice the maximum size aggregate, to avoid the potential for a continuous flow path along the aggregate.

This cell is designed to accommodate specimens taken from either cylinders or cores using a recessed rim to create a watertight seal, and exposing a 75-mm diameter surface. The reservoir capacity of the cells employed was approximately 720 mL.

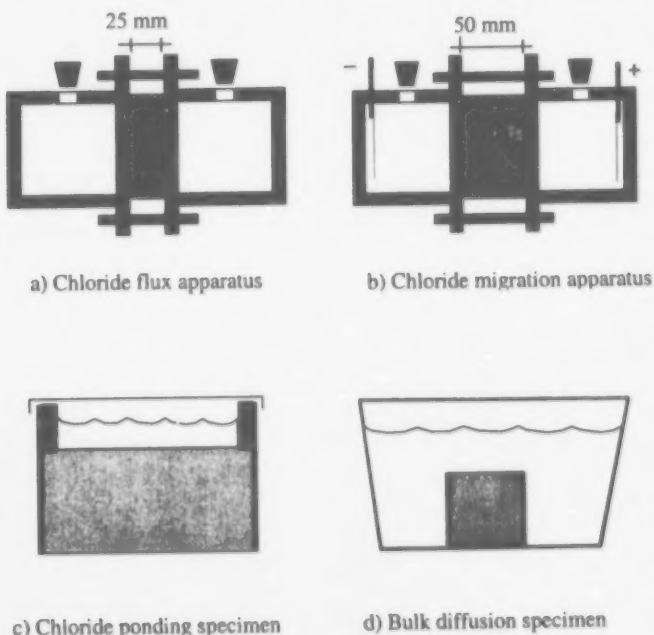


Fig. 1: Schematic representation of diffusion coefficient test methods

The NaOH reservoir is monitored for chloride breakthrough using chloride titrator strips on a monthly basis. Once chloride breakthrough occurs, 10-mL aliquots are removed from both reservoirs and titrated for chloride concentration. Aliquots are then removed and titrated monthly until at least six consecutive diffusion coefficient calculations agree to within $\pm 1 \text{ mm}^2/\text{yr}$. The exposed surface is centered in the reservoir, so the aliquots do not lower the solution level enough to expose the concrete surface.

The effective diffusion coefficient, D_e , is calculated from the chloride concentration measurements using Fick's second law of diffusion (Eq.1). The calculation is simplified using Eq. 2.¹ Since solution concentrations, the specimen thickness, and time of exposure are known, the effective diffusion coefficient is determined by setting the Ratio in Eq. 2 to 1.000.²

$$C_{x,t} = C_a + (C_{se} - C_a) \left(1 - \operatorname{erf} \left[\frac{x}{2\sqrt{D_e t}} \right] \right) \quad (1)$$

$$\text{Ratio} = \frac{\left(\frac{C_{x,t} - C_a}{C_{se} - C_a} \right)}{1 - \operatorname{erf} \left[\frac{x}{2\sqrt{D_e t}} \right]} \quad (2)$$

Where:

- $C_{x,t}$ = The chloride concentration in the NaOH reservoir at time t , ppm
- C_{se} = The chloride concentration in the NaCl reservoir, ppm
- C_a = Background chloride concentration, ppm
- x = Sample thickness, mm
- erf = The error function
- D_e = The effective chloride diffusion coefficient, mm^2/yr
- t = Time of exposure in cell, yr

The chloride flux test is fundamentally the most sound procedure to determine the diffusion coefficient, since the only driving force is a concentration gradient. Also, since the specimen is primarily a membrane, chloride binding does not skew the calculation. However, it is a real-time test that can take a couple of years to complete for high quality concrete. Practically, the usefulness of this procedure is limited to long-term research.

¹ In this study, C_a is set to 0.0 because the NaOH solution was made with de-ionized water. If the initial NaOH solution contained Cl^- , use the net concentrations for C_{se} and $C_{x,t}$.

² In practice, the trial and error process is simplified by using the Goal Seek or Solver functions of an Excel spreadsheet.

3.2.2 Migration methods

Both chloride migration procedures use the same basic cell design as the chloride flux test except that the concrete slice is thicker, and the reservoirs contain mesh electrodes. The application of a modest electrical potential (10 to 15 VDC) accelerates the migration test from a multi-year test to a few weeks, even with a larger specimen thickness. The authors chose to modify the migration cell commonly employed in the literature by using an anode electrode (+) made from platinized niobium mesh instead of stainless steel to minimize cell maintenance resulting from anode corrosion after chloride break-through.

Migration procedure 1 is a modified version of Streicher and Alexander's (1995) chloride conduction procedure. The modification is simply that the concrete is saturated with chloride by applying the electrical potential, instead of vacuum saturation. After polarization the reservoir solutions are emptied, and both sides are refilled with fresh 2.5 M NaCl solution for the conductivity measurement. Fig. 2 shows that the modification saturates the sample with chloride prior to the measurement, validating the calculation assumption that the pore solution conductivity is known.

The diffusion coefficient is calculated from conductivity measurements using Eq. 3 below. The fundamental assumption behind this calculation is that the pore solution conductivity is known. For most concrete the pore solution conductivity is not known, so preconditioning is required to saturate the sample with a known chloride solution.

For this technique, a 2.5 M concentration of NaCl solution was selected to saturate the pore solution. Although the 2.5 M solution exceeds the 0.5 M limit of the Debye-Huckel theory for chloride transport, the larger concentration was chosen to minimize the error induced by other ions solvating into the pore solution from the concrete. Therefore, the pore conductivity and diffusivity numbers can be assumed to be that of a 2.5 M Cl^- solution, and any charge carried by ions solvating from the concrete (i.e., K^+ , Na^+ , Ca^{2+} , SO_4^{2-} , OH^-) can be considered negligible. Please refer to the original work by Streicher and Alexander (1995) for a more detailed discussion of the method, assumptions, and calculation procedures.

$$D_c = 31536 * (10)^{13} D_o \frac{\sigma_c}{\sigma_o} \quad (3)$$

Where:

$3.1536 * (10)^{13}$ is the conversion factor for m^2/s to mm^2/yr

D_c = Diffusion of chloride ions through concrete pores (mm^2/yr)

D_o = Diffusion of chloride ions through 2.5M NaCl solution ($0.165 \text{ m}^2/\text{s}$)

σ_c = Conductivity of concrete (calculated from voltage drop and current measurements on cell), S/m

σ_o = Conductivity of solution (16 S/m for 2.5 M NaCl)

Migration procedure 2 is the accelerated chloride flux procedure presented by Zhang and Gjorv (1994 and 1995). Calculation of the diffusion coefficient in this method is based on the drift velocities of chloride ions within concrete. Zhang assumed that the drift velocity of chloride ions is affected by relaxation and

electrophoretic effects of the ionic cloud, and by a concentration related retardation in ion migration velocity. By taking these assumptions into consideration, they proposed that the effective diffusion coefficient could be calculated as described in Eq. 4.

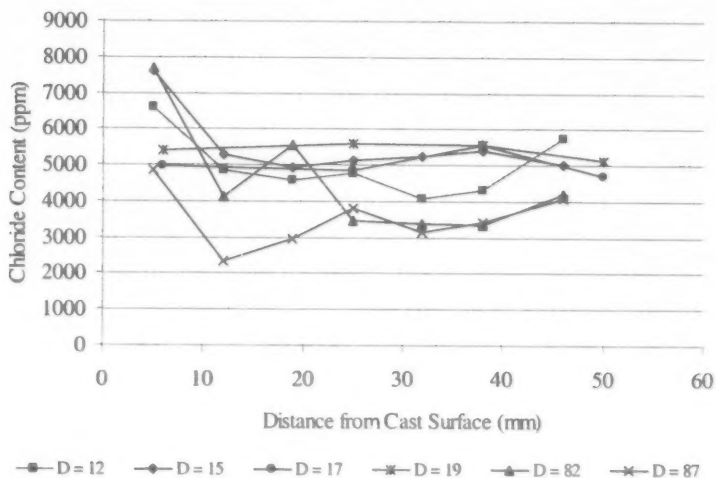


Fig. 2 - Chloride content of specimens after "Migration procedure 1"

In this procedure, the concentration of chloride must remain less than 0.5 M for the Debye-Huckel theory to be satisfied, thereby making Eq. 4 valid. In practice, the constants β (Volt/K^o) and α (cm²·L^oK/mol·Volt) are calculated for each specimen, and the change in chloride concentration over time in the anodic chamber is monitored to obtain dC/dt . The effective diffusion coefficient is then calculated using Eq. 4. The original work by Zhang and Gjorv (1994) provides a detailed discussion of the derivation of the equations, assumptions, and calculation procedures.

A practical limitation of this procedure is that it is laborious, requiring daily monitoring and chloride sampling, so it was discontinued after use in two evaluations.

$$D_e = 3.1536 \cdot (10)^9 \beta \alpha \frac{dC}{dt} \quad (4)$$

Where:

$3.1536 \cdot (10)^9$ is the conversion factor from cm²/s to mm²/yr

$$\beta = \frac{300kV^o}{ze_o V_{act}}$$

$$\alpha = \frac{T}{\Delta\Psi} \frac{LV}{c_o A_o}$$

D_e = Effective chloride diffusion coefficient, mm^2/yr

A_o = Cross section area, cm^2

c_o = Source chloride concentration, mmol/cm^3

e_o = Charge of a proton, $4.8 (10)^{10}$ esu.

k = Boltzmann constant, $1.38 (10)^{16}$ erg./K·ion

L = Distance between electrodes, cm

T = Absolute temperature, K

V = Volume of collecting cell, cm^3

V^o = Drift velocity of at infinite dilution, (see reference)

V_{act} = Actual net drift velocity for ions in solution, (see reference)

z = Ionic valance

$\Delta\Psi$ = Applied electric potential, Volts

$\frac{dC}{dt}$ = Rate of change in chloride concentration over time

3.2.3 Chloride ponding

The chloride ponding specimens were 150-mm diameter by 150-mm high concrete cylinders. The bottom half of a 150- x 300-mm cylinder mold was used to cast the specimen and the top half of the mold was used to form the ponding dike. Specimens were cast, stripped, moist-cured for 28 days, dried for one week, and had the sides covered with two coats of epoxy paint before the dike was caulked in place. After the silicone caulk dried, the dike was filled with 2.5 M NaCl solution and then covered by a plastic bag. The 150- x 150-mm chloride ponding specimens were stored on racks and routinely monitored for evaporation and leaks.

Individual ponding specimens were cast for each exposure period of interest. At the end of each exposure period the dike and any silicone remnants were removed, the cylinder was rinsed, and then sliced in 6-mm increments. Each slice had the epoxy-coated edge removed, and was carefully dried, pulverized, and analyzed for the acid soluble chloride content according to ASTM C 1152 and AASHTO 260.

The apparent diffusion coefficient, D_a , and effective surface concentration were calculated from the chloride content profile using Eq. 1 and a nonlinear regression routine. The background chloride content, C_b , was determined for each evaluation from initial chloride content measurements taken from specially cast cubes or compressive strength specimens.

3.2.4 Bulk diffusion

Bulk diffusion differs from the ponding test (3.2.3) in that the specimen is coated with epoxy on all but the finished surface, is saturated with water prior to chloride exposure, and is totally immersed in a chloride solution. The acid soluble chloride profile is then obtained from powder samples taken at 1-mm increments after the exposure period. The diffusion coefficient is calculated by fitting the chloride profile to Eq. 1. The test conditions force chloride ingress to occur only by diffusion,

whereas conditions in the ponding test allow combined diffusion, sorption, and wicking.

4 Results

Table 1 summarizes diffusion coefficient estimates from six individual experiments. Evaluations A through E were included in analysis of variance. Evaluation F was provided only for illustrative purposes and was not included in the statistical analysis.

Fig. 3 illustrates the effect of chemical and mineral admixtures on the diffusion coefficient using three different test methods. Each bar represents the pooled mean value of within-evaluation comparisons from Table 1, using evaluations A through D.

It is important to note that the comparisons shown in Fig. 3 are not isolated experimental results, but are the pooled results from four individual experiments. The trends indicated in the graphs are repeatable, as each individual experiment produced similar results, even though mixture proportions differed.

To illustrate, the diffusion coefficients pooled for the Reference shown in the left graph are Mix numbers 1, 3, 4, 5, 9, 10, 11, 15, and 16. The comparable OCI Mix numbers are 2, 6, 7, 8, 12, 13, 14, 17, and 18, respectively.

Similarly, the values pooled together for the Reference shown in the right graph are from Mix numbers 3, 6, 9, 12, 15, and 17. The 4% SF values are the mean values from Mix numbers 4, 7, 10 and 13. The 8% SF values come from Mix numbers 5, 8, 11, 14, 16, and 18.

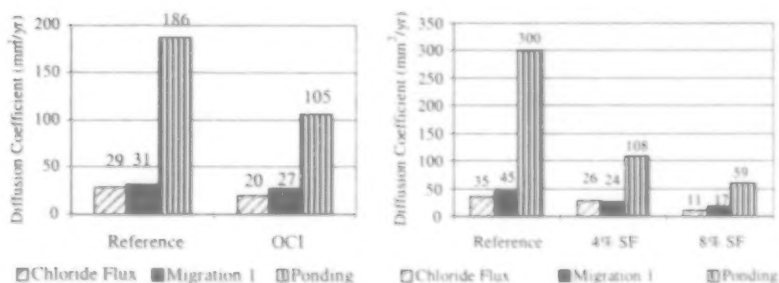


Fig. 3: Comparison of admixture effect across test methods

5 Discussion

The goal of any engineering property test is to produce unbiased parameter estimates useable in subsequent calculations. Therefore, an appropriate test method must adhere to the same boundary conditions as the calculation procedure. In the case of chloride diffusion coefficient tests, these boundary conditions dictate that the influence of alternative chloride transport mechanisms be minimized.

Since chloride diffusion occurs through the water filled pores in concrete, saturation and minimal water transport are essential conditions for an appropriate test procedure. This requirement is satisfied in the saturated conditions of the chloride flux, migration, and bulk diffusion test methods, but is lacking in the chloride ponding test.

The data presented in Table 1 clearly show that the flux, migration, and bulk diffusion methods produce similar parameter estimates, whereas the historical approach of fitting chloride profiles is heavily biased. This bias is most probably caused by chloride ingress through alternative transport modes.

Water and chloride transport through concrete is influenced by hydraulic pressure and/or saturation gradients produced by wetting and evaporation fronts. A saturation gradient caused by evaporation near the dry bottom surface of the ponding test pulls *both* water and chloride through the concrete. This combined water and chloride transport mode is termed wicking. Buenfeld et al. (1995) showed that wicking can be a significant chloride transport mode and should be modeled separate from diffusion.

Wicking and other transport mechanisms are included in the apparent diffusion coefficients calculated from both structures and chloride-ponding tests, but are absent in the effective diffusion coefficients calculated in the other methods presented. This disparity is clearly illustrated in Fig. 3. Furthermore, if the test conditions eliminate water transport through wicking and sorption, as in the bulk diffusion test, the apparent diffusion coefficients obtained by fitting chloride profiles are reasonable.

For example, Mix 22 has comparable mixture proportions to Mix 11, but the apparent diffusion coefficients do not agree. In contrast, Mix 22's apparent diffusion coefficient is more in line with the effective diffusion coefficients calculated from the chloride flux and migration methods for Mix 11. This example should not be construed as a general statement of fact, as it is just a single observation. However, the observed response is consistent with the theories advanced above.

To draw scientifically correct inferences on the data shown in Table 1, analysis of variance (ANOVA) was used. Because separate mixes were not made when running each test method, there are between-mix and within-mix comparisons for this split-plot design. For example, the evaluation, corrosion inhibitor dose, and silica fume effects are between-mix comparisons. However, the test method effect is a within-mix comparison. Due to the unbalanced nature of this split-plot design detailed discussion of the formal data analysis is beyond the scope of this paper. (Milliken and Johnson 1984).

Table 1: Chloride diffusion coefficient summary table.

Eval. #	Mix #	Mix Code	Measured Diffusion Coefficient (mm ² /yr)					Surface Cse (ppm)	Background Co (ppm)	Age t (yr)
			D _e - Flux	D _e - Migr. 1	D _e - Migr. 2	D _a - Pond				
A	1	362-38-0	27	19		192		5250	500	2.0
A	2	362-38-0.5	25	17		84		3503	500	2.0
B	3	307-50-0	59	56		980		3626	550	1.5
B	4	307-50-4	48	39	60			9814	550	1.5
B	5	307-50-8	14	19	33			6773	550	1.5
B	6	307-50-0.5	39	45	31			4650	550	1.5
B	7	307-50-4.5	42	27	21			8111	550	1.5
B	8	307-50-8.5	15	20	24			5580	550	1.5
C	9	390-38-0	45	40	38			14326	550	2.3
C	10	390-38-4	14	26	36			14192	550	2.3
C	11	390-38-8	10	18	31			12038	550	2.3
C	12	390-38-0.5	18	30	30			12596	550	2.3
C	13	390-38-4.5	9	20	16			17198	550	2.3
C	14	390-38-8.5	8	13	22			17932	550	2.3
D*	15	380-40-0	30	45				8626	660	2.0
D*	16	380-40-8	11	17				7652	660	2.0
D*	17	380-40-0.5	18	54				8867	660	2.0
D*	18	380-40-8.5	6	14				7661	660	2.0
E*	19	356-40-0		19						
E*	20	356-40-0.5		18						
F**	21	400-33-0								
F**	22	432-37-8								

* The chloride profiles for these evaluations came from specimens in a severe marine exposure

** Bulk diffusion test - 28 days in 5M NaCl solution at 40 °C; coefficient corrected for 21 °C (see Amey et al., 1998)

Statistical analysis of the data in Table 1 shows that the estimates obtained using the chloride flux and migration methods are essentially equivalent. The analysis also shows the reduction in the diffusion coefficients from chemical and mineral admixtures are significant. Replacing 4-8% of cement with silica fume produced a dramatic reduction in the diffusion coefficient regardless of the testing method. This reduction was in excess of 50% for 8% replacement. Furthermore, the organic corrosion-inhibiting admixture reduced the diffusion coefficient by 10 to 30%.

6 Conclusions

1. The flux and migration test methods presented produce similar values for the effective chloride diffusion coefficient for concrete.
2. Fitting chloride profiles from ponding or environmental exposure experiments to Fick's second law of diffusion can produce apparent diffusion coefficients 2 to 6 times greater than the effective diffusion coefficient.
3. Statistical analysis of the data presented indicates that all the test methods show a progressive reduction in the diffusion coefficient with increasing silica fume replacement. Also, significant reductions in the diffusion coefficient occur when an organic corrosion-inhibiting admixture was added to the concrete.

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34 PREDICTION OF CORROSION RESISTANCE OF CONCRETE STRUCTURES

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Abstract

Recently, large-scale concrete structures exposed to severe environment are increasingly built in various locations. The corrosion may severely affect the durability and service life of such a concrete structure. It is, therefore, necessary to develop durable concrete to enhance the corrosion resistance. The corrosion resistance of concrete can be identified through accelerated corrosion test. The purpose of the present paper is, therefore, to devise a reasonable and accurate method to predict the amount of corrosion of reinforcing steels. The proposed method which is basically based on the concept of Faraday's Law, determines accurately the corroded amount of a rebar according to accelerated corrosion time. The corrosion is accelerated by employing the potentiometric corrosion test arrangement. The effects of admixtures in concrete including blast furnace slag and silica fume have been also studied to explore the relative corrosion resistance of concrete. The present study indicates that the cement content of concrete and use of mineral admixtures have a significant influence on the corrosion resistance of reinforcement. The bond strength is found to decrease with increasing corrosion rate. The bond strength drops to half of initial bond strength at 15% corrosion rate. The proposed method allows the prediction of accurate corrosion amount in reinforcing steel and the estimation of service life of actual concrete structures.

Keywords: Accelerated corrosion test, corrosion of steel bar, endurance life, mineral admixtures, severe environments.

1 Introduction

Concrete is one of the most promising construction materials and has been widely used in all types of civil and architectural structures. As the development of infrastructure becomes very active in most countries, large-scale construction is being taken place in land, seaside, underground and ocean area. These structures often face very severe environments. The corrosion of reinforcing steels is one of the most deleterious factors that affect the safety of structures. The corrosion of steel bars in concrete is of great concern in structural engineering area. However, accurate method for the quantitative analysis of corrosion has not been established yet.

The corrosion resistance of concrete can be identified through accelerated corrosion tests. However one major problem of the accelerated corrosion tests lies in the fact that one cannot properly estimate the actual corrosion rate of real structures from the laboratory acceleration test results. In other words, it is difficult to correlate quantitatively the accelerated test results to the actual corrosion amount of a real structure.

The purpose of the present paper is, therefore, to devise a reasonable and accurate method to predict the amount of corrosion of reinforcing steel and to analogize this accelerated test result to actual structures.

The proposed method is basically based on the concept of Faraday's Law, which states that, in an electrochemical cell, the numbers of moles of reactants and products are proportional to the electrical charge passed through the external circuit and that an equivalent mass of each substance is produced or consumed.

In the present study, the corrosion is accelerated by employing the potentiometric corrosion test arrangement. The proposed method can determine exactly the corroded amount of reinforcing steel by mass according to the accelerated corrosion time. The effects of various mineral admixtures in concrete including silica fume and blast furnace slag have been also studied to explore the relative corrosion resistance of concrete.

2 Mechanism of corrosion development

2.1 Presence of chloride in concrete

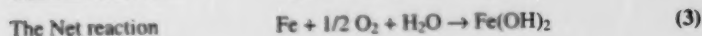
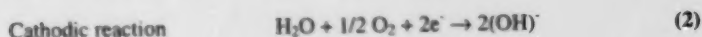
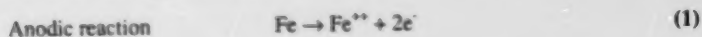
Presence of chloride in concrete can be divided into two types. The first source of chloride is from using sea sand or chemical admixtures containing chloride. The other source of chloride comes from direct penetration of seawater when concrete structures are constructed in seaside or offshore area.

2.2 Mechanism of corrosion initiation of steel bars

It is well known that the passivating very thin film is formed at the steel-concrete interface in reinforced concrete structures.

The protective action of passivating layer is of essentially an electrochemical nature and is guaranteed by the high alkalinity of concrete ($\text{pH} = 12.5\text{--}13$). However,

the presence of depassivating chloride in large enough amounts at passivating layer may destroy locally the film. As the film is destroyed, the steel in concrete is corroded by the occurrence of corrosion current between steel and concrete interface, namely, due to potential difference between anode and cathode.



3 Proposed corrosion method

3.1 Outline

There exist many methods that are used for the corrosion test of steel bar such as by immersing in seawater or by raising temperature of the specimen. These methods give rough evaluation for the corrosion, but they do not give any quantitative results for the corrosion amount. The present study, therefore, devises a new method that can yield an accurate amount of corrosion and the time to corrode a certain specific level by employing so-called the accelerated potentiometric corrosion test. The proposed corrosion test method gives the current versus time curve for a specific test specimen with reinforcing steel bar. The detail procedure of the proposed test method is given in the following section.

3.2 Materials and test specimens

The main principle in selecting the test materials is to see the effects of types of cement and mineral admixtures. Since it is known that the use of admixtures may influence the corrosion resistance of concrete, several mineral admixtures including silica fume and blast furnace slag have been used in the present study.

Table 1 shows general characteristics of cement used and Table 2 summarizes the mixture proportions of various test series. Here, N1 represents the normal strength concrete with ordinary cement and H1 represents the high strength concrete with ordinary cement.

SF represents silica fume and BS represents blast furnace slag. The last value of the test series name is the amount of air entraining agent.

Table 1: Characteristics of the cement

Specific Gravity (wt^3)	Expansion Autoclave (%)	Surface Area (cm^2/g)	Size (μm)		Setting time (hr : min)		Compressive Strength (kg/cm^2)			
			Max	Mean	Initial	Final	1day	3day	7day	28day
3.144	0.1	3200	100	21	4:20	6:40	90	200	285	375

The fine aggregates are from the river near Seoul, and the specific gravity and absorption rate are 2.60 and 1.5 %, respectively. The coarse aggregates are crushed stones, and the specific gravity and absorption rate are 2.75 and 1.0 %, respectively.

Blast furnace slag is manufactured from Po-Hang Iron Works and its surface area is approximately 4000 cm²/g. Silica fume is from the United States of America and its surface area is about 200,000 cm²/g.

The specimens for corrosion tests are made from the cylinders of size $\phi 100 \times 200$ mm. The reinforcing bars are embedded in the cylinder with the cover depth of 1 cm or 2 cm. The centrally embedded rebar specimens have been used for bond test after a certain amount of corrosion has been taken place in the steel bar of the concrete specimen.

Table 2: Summary of test specimens

Variables of test	Cement (kg/m ³)	Mineral Admixtures (kg/m ³)	Water (kg/m ³)	Fine aggregate (kg/m ³)	Coarse aggregate (kg/m ³)	Air Entraining agent (g/m ³)
N-0-0	350	0	190	740	1060	0
N1-0-0	380	0	164	850	1040	0
N1-0-0.15	380	0	164	850	1040	570
N1-SF10-0	342	38	164	850	1040	0
N1-SF20-0	304	76	164	850	1040	0
N1-SF10-0.15	342	38	164	850	1040	570
N1-BS12.5-0	332.5	47.5	164	850	1040	0
N1-BS25-0	285	95	164	850	1040	0
N1-BS12.5-0.15	332.5	47.5	164	850	1040	570
N1-SF10,BS25-0	247	133	164	850	1040	0
H1-0-0	550	0	154	708	1771	0
H1-0-0.15	550	0	154	708	1771	570
H1-SF10-0	495	55	154	708	1771	0
H1-BS12.5-0	481.25	68.75	154	708	1771	0
H1-SF10,BS25-0	357.5	192.5	154	708	1771	0

3.3 Experimental procedure

3.3.1 Procedure of accelerating steel corrosion in concrete

The corrosion circuit to accelerate steel corrosion in concrete is described in Fig. 1. After 28 days of curing in moist condition, the test specimen was partially immersed in 5 % NaCl solution and an electric circuit with 20 V D.C. power supply is connected as shown in Fig. 1.

The steel bar embedded in concrete is connected with "+" of D.C. power supply and the naked bar (the cathode) is connected with "-" of it. As the steel corrosion progresses, the value of measured current becomes larger. The accumulation of corrosion makes the bar to expand, the cracks are developed, and eventually the concrete cover is spalled off. The automatic measuring data logging system was used to measure the current at each time step after the corrosion is started. The current-time graph is then automatically plotted on the sheet.

The area under the current-time graph is directly related to the total amount of steel corrosion according to Faraday's Law. This is the main feature of the present

corrosion study.

In order to measure the current in natural condition without connecting electric circuit the accelerated corrosion circuit arrangement has been disconnected for a short time at some time interval and the current in the specimen has been measured from the ammeter with the precision of 0.01 mA.

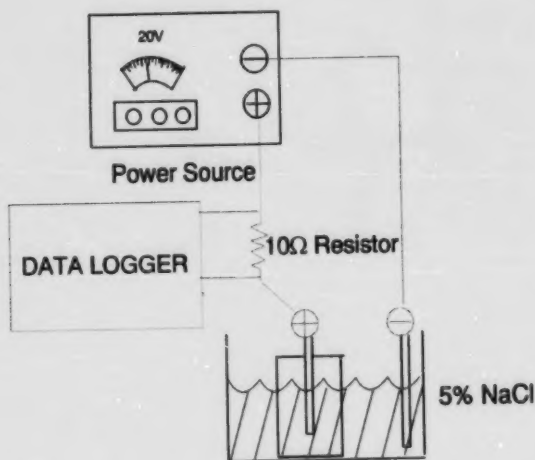


Fig. 1: Arrangement of accelerated steel corrosion test

3.3.2 Test of bond strength

The bond strength is one of the most important factors that are affected by corrosion of steel bars. Since the present study enables one to make the bar corrode by specific amount from the proposed test method, the variation of bond strength has been extensively studied according to the corrosion amount of a steel bar. This can give a quantitative reduction of bond strength due to the corrosion of steel bars. This kind of research result has not been reported yet and may give very important basis for the bond strength design of concrete structure which are subjected to corrosion environments.

4 Analyses of the results

The present theory for corrosion comes from the fact that the steel corrosion is an electro-chemical reaction. The proposed theory is based on Faraday's Law saying that total passed charge is preserved during the chemical reaction and the total current from accelerated corrosion test is equal to the total current from natural corrosion.

Namely, if one can measure the current in natural condition, it is possible to

calculate the corrosion time for actual structures from the condition that the total currents from accelerated test during short period are equal to the total currents for actual structures for long period.

This can be done by dividing the total area of current-time curve of accelerated test by the current of concrete specimen in natural condition.

4.1 Results of compressive strength

The results of compressive strengths for each series are given in Table 3.

Table 3: Compressive strength

Test series	28day Compressive strength (MPa)
N-0-0	30.0
N1-0-0	34.7
N1-0-0.15	34.3
N1-SF10-0	42.3
N1-SF20-0	57.0
N1-SF10-0.15	42.5
N1-BS12.5-0	37.2
N1-BS25-0	31.1
N1-BS12.5-0.15	34.3
N1-SF10,BS25-0	46.7
H1-0-0	55.6
H1-0-0.15	51.5
H1-SF10-0	77.4
H1-BS12.5-0	58.7
H1-SF10,BS25-0	69.4

4.2 Results of corrosion test

The corrosion test results are shown in Fig. 2 and Fig. 3 in terms of current versus time curve. The measurements have been automatically done by data logging system and plotted on the graphical paper. It can be seen from these figures that the currents are not increased up to a certain time and then increase rapidly. (Fig. 2)

The rapid increase of currents indicates that the corrosion accumulate very rapidly due to the cracking around the steel bar. Fig. 2 indicates that the current-time curves from the corrosion tests are very different among various mixtures of concrete. The plain mixture without any mineral admixtures (N1-0-0) shows very short resistance to corrosion and the service life will be relatively short.

However, the test series with silica fume (SF) exhibits very high resistance to corrosion. Fig. 3 also shows the corrosion resistance for various mixtures with low water-cement ratio, i.e., high strength concrete mixture. It is obvious from these figures that the high strength concrete mixtures show enhanced resistance to corrosion, compared with normal strength concrete.

The area under current-curve represents the corrosion amount based on the Faraday's Law.

The integral of the current-time curve shows the corrosion amount.

According to Faraday's Law, 1 mole of steel uses 2 F (1 F=96,500 C, 1 C =1 Ampere $\times$ 1 second) and 1 mole of steel is 55.847 g.

So, it is possible to estimate the corrosion amount of steel reinforcement embedded in concrete according to following equation, in which the corrosion amount is expressed as the number of moles of steel molecule, and q is the passed current at a specific time in ampere.

$$\text{Corrosion amount (mole)} = \frac{1}{2 \times 96,500} \int q \, dt \quad (4)$$

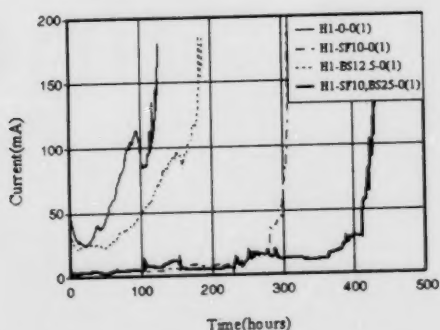


Fig. 2: Resistance curve according to use of mineral admixtures

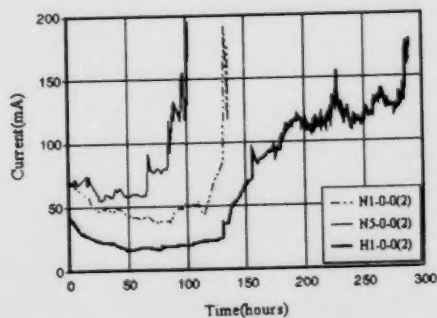


Fig. 3: Resistance curve of corrosion according to the type of cement

4.3 Results of bond strength

To explore the relation between steel corrosion and bond strength, the test specimens are enforced to be corroded to a certain predetermined corrosion amount. This can be possible because the proposed method yields exactly the specified amount of corrosion. The bond test is performed by automatic controlled MTS test machine and the results are summarized in Fig. 4 and Fig. 5.

The bond strength are decreased as the corrosion accumulates. The bond strength decreases down to half of the original bond strength when the corrosion rate reaches about 15 %, and the bond strength completely vanishes when the corrosion rate is more than 25~30 %.

The degrading curve of bond strength due to common is shown in Fig. 5.

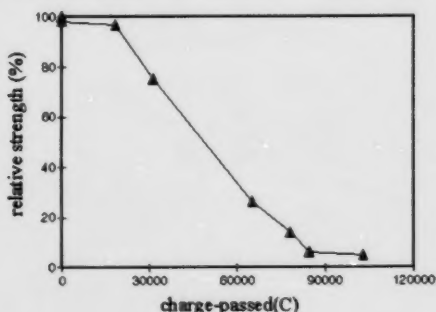


Fig. 4: Relative bond strength according to total charge passed

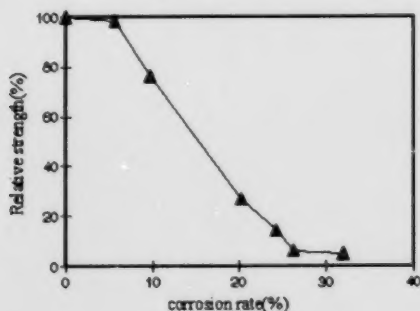


Fig. 5: Relative bond strength according to corrosion amount

4.4 Endurance life

The present study allows the calculation of endurance life or failure life due to corrosion of steel for a certain mixture concrete. This can be done by equalling the total currents from accelerated test to the total currents from natural environmental condition. Since the current in natural condition is much smaller than that under accelerated test, one should equate the total area up to the failure point as shown in Fig. 6 and Fig. 7.

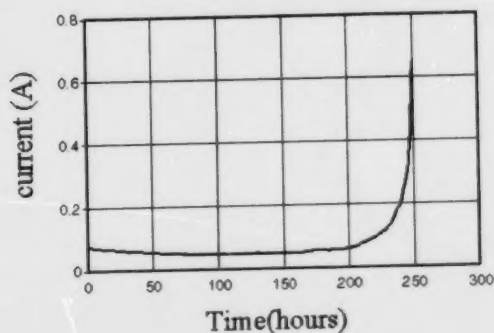


Fig. 6: Corrosion Resistance curve. (N-0-0)

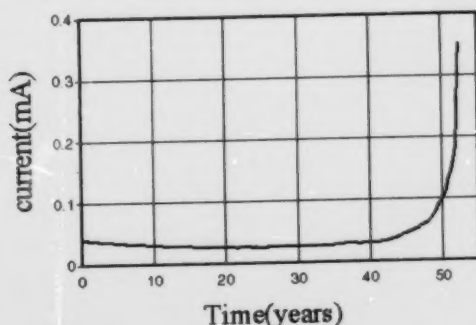


Fig. 7: Prediction of service life for corrosion failure. (N-0-0)

The total passed charge of the specimen from the accelerated test is given in Fig. 6. The total charge is 65,121.64 Coulomb (Fig. 6) and molecular weight of steel is 55.847 g, and 193,000 Coulomb ($96500 = 2 \text{ C}$) is required to react the steel corrosion. Initial weight of steel is 93.33 g. This is 1.67 mole and corresponds to 322,547.91 C. Consequently, the specimen corroded about 20 % by volume of steel. The measured current in natural condition is 0.36 mA at the end of test. The

maximum bond strength of this specimen decreased down to 26 % of initial bond strength at 20 % loss of volume of steel due to corrosion. Therefore, corrosion is a major factor that reduce the bond strength considerably.

5 Conclusion

The purpose of the present study is to devise a new method that can predict the corrosion amount of a steel bar in concrete accurately by employing the potentiometric corrosion test method. The following conclusions can be drawn from the present study.

1. A reasonable test arrangement that can predict the corrosion amount of steel embedded in concrete is proposed. The corrosion amount is calculated on the basis of Faraday's Law.
2. The endurance life of concrete structures can be estimated by equating the total currents from accelerated test to the currents from natural condition, and the procedure was established in this study.
3. The present study indicates that the cement content of concrete and use of mineral admixtures have a significant influence on the corrosion resistance of reinforcement in concrete. The silica fume concrete has good resistance to corrosion and the combination of silica fume and slag gives best performance to corrosion.
4. To survey the relation between reinforcement corrosion and bond strength, the bond strength tests have been carried out and the bond strength tends to decrease with increasing corrosion rate. The bond strength drops to half of the initial bond strength when the corrosion rate reaches 15 % (15 % volume of corrosion).

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35 CORROSION MONITORING USING A NEW SENSOR SYSTEM FOR INSTALLATION INTO EXISTING STRUCTURES

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Abstract

Since recent years modern sensor systems are used to monitor the corrosion risk for the reinforcement of concrete structures, i.e. the depth of the critical chloride content initiating corrosion. The Anode-Ladder-System has e.g. been used successfully since 1990 world-wide in structures exposed to aggressive environment. As these systems have to be installed before placement of the concrete, the new Expansion-Ring-System has been developed, which can be installed into drilled holes of existing structures. It consists of 6 anode rings within the main sensor and a cathode bar, both with special expansion mechanisms. Using this sensor system the time-to-corrosion can be determined continuously, enabling the owners of buildings to initiate preventive protection measures before damages due to corrosion of the reinforcement like cracks and spalling have occurred.

Keywords: Corrosion, monitoring, steel, concrete, reinforcement, anode, cathode, sensor

1 Introduction

Corrosion of the reinforcement has become a major problem world-wide, especially for structures exposed to aggressive environments such as bridges, parking decks, tunnels, offshore structures or other buildings exposed to seawater or deicing salts. As result, the repair costs nowadays constitute a major part of the current spending on the infrastructure.

Many new systems and materials have been developed to repair the damages and to increase durability, e.g. high performance concretes. However, there is only limited long-term experience with these developments. Therefore sensor systems

are needed enabling the owners of the structures to monitor the corrosion risk and to take protective measures *before* the damaging processes start.

As known, the costs for repair measures increase more than proportionally with time, when the reinforcement already has started to corrode. However, the traditional monitoring systems like reference cells only show the local onset of corrosion, but not the time-to-corrosion, which is measurable using the Anode-Ladder-System or the new Expansion-Ring System.

2 Basic principle to determine the time-to-corrosion

The basic measuring principle is to place electrodes into different depths related to the concrete surface and to measure the onset of corrosion of these electrodes one by one (see Fig. 1).

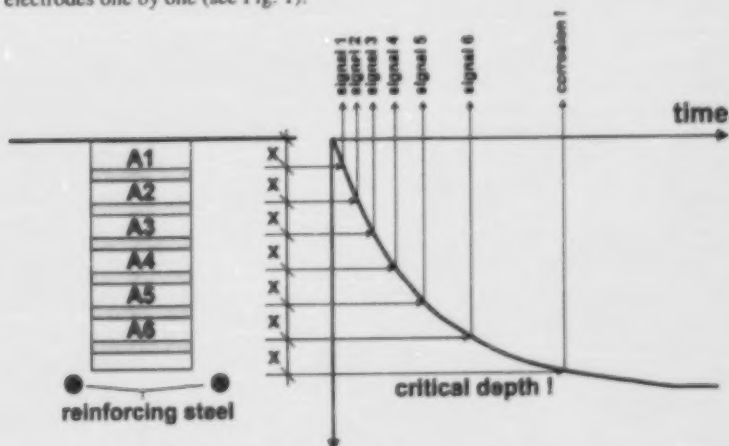


Fig. 1: Basic principle to determine the time-to-corrosion

The measuring electrodes are made of steel with a similar composition as reinforcing steel to ensure that they will start to corrode at the same time, when a rebar at the same depth would start to corrode. Comparative tests have been carried out at the Institute for Building Materials Research of the RWTH Aachen, showing that there was no significant difference in the corrosion behaviour of the steel used for the measuring electrodes (anodes) and reinforcing steel with different degrees of pre-rusting before installation into the concrete specimens.

Using adequate calculation models the time to corrosion can be determined at any time provided that the cover thickness of the reinforcement is known. If necessary the concrete cover can be measured using covermeters.

3 Development of the expansion-ring-system

The development of the corrosion monitoring systems has been carried out based on an extensive research program on chloride-induced corrosion of steel in concrete at the Institute for Building Materials Research of the Technical University of Aachen. More than 500 concrete specimens with different concrete compositions have been investigated under varying environmental conditions using macrocell current measurements as testing method (see e.g. [1]-[5]).

The tests have shown, that this technique can also be used to monitor the ingress of aggressive substances like chlorides or carbonation by measuring electrical signals between different steel bars installed at defined locations.

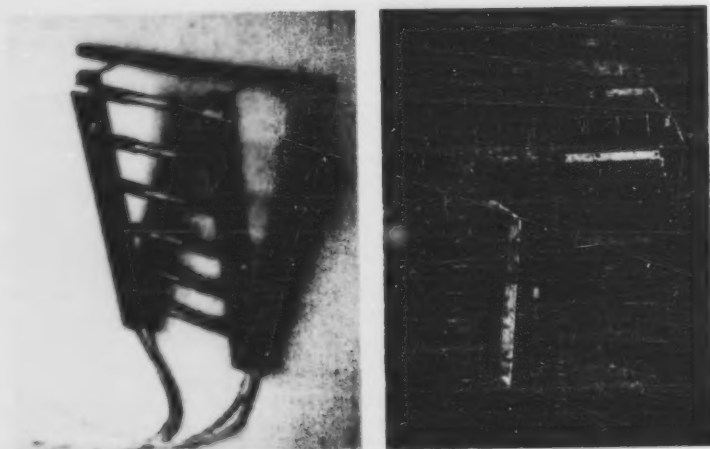


Fig. 2: Photographs of the Anode-Ladder-System to monitor the corrosion risk of the reinforcement installed during construction

Since 1990 the so called Anode-Ladder-System (see Fig.2) has been used world-wide in different types of structures to monitor the corrosion risk for new reinforced concrete structures in aggressive environment [6]. More than 600 Anode-Ladders have been installed into tunnels, bridges, foundations and other structures exposed to aggressive environment in Germany, Austria, Denmark, Hong Kong, Japan, Croatia, Egypt, Switzerland, The Netherlands, Sweden, Taiwan and Australia.

The six steps of the ladder made of steel will be depassivated one after the other by chlorides or carbonation of the concrete showing the actual critical depth, where the reinforcement is not any more protected by the passive layer induced by the alkalinity of the pore solution of the concrete.

This sensor can only be installed before or during placement of the concrete into new structures. As further development the Expansion-Ring-System is able to monitor the corrosion risk also for existing structures by inserting the sensors into drilled holes.

4 Description of the expansion-ring-system

The Expansion-Ring-System consists of the Expansion-Ring-Anode and a cathode-bar. Both electrodes are inserted into holes, which have to be drilled into the concrete surface. The holes for the Expansion-Ring-Anode shall not be drilled by hand without fixed drilling equipment, because the geometry of the holes has to be very exact. Usually the hole to be drilled to fix the drilling equipment by a dowel is later used to insert the cathode-bar into the concrete. Fig. 3 shows the installation- and measurement-procedure schematically.

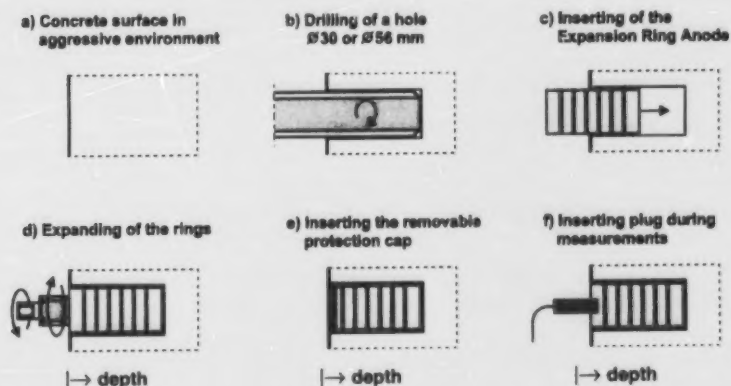


Fig. 3: Installation procedure of the new Expansion-Ring-Anode

The main measuring sensor is the Expansion-Ring-Anode (see Fig. 4, (A)). Similar to the six bars of the Anode-Ladder-System [6] the Expansion-Ring-Anode consists of six measuring rings in different distances from the concrete surface, in 1 cm-steps from 1 to 6 cm. In this way the ingress of chlorides and/or carbonation from the concrete surface into the concrete and the subsequent corrosion risk of the reinforcement can be measured as it was presented in section 2. Above, between and below these metal rings altogether seven insulation rings are established to seal the area between the rings, i.e. to prevent water or chlorides penetrating along the drilled surface deeper into the hole of the Expansion-Ring-Anode.

The cathode (see Fig. 4, (B)) consists of two pieces of a platinum-coated bar to be expanded by a special mechanism to ensure a durable contact to the concrete.

The Expansion-Ring-Anode and cathode bar are connected to the concrete by turning nuts at the upper part of the sensors, leading to an expansion of the rings in a way that both sensors can be tightened sufficiently strong. During installation of the sensors the torque of the sensors will be measured to ensure that the tension between sensor and concrete is sufficient for a durable contact and not too high to induce problems of cracking of parts of the sensor or the concrete. To prevent cracking of the concrete, the sensors should not be installed at edges or near the ends of concrete elements.

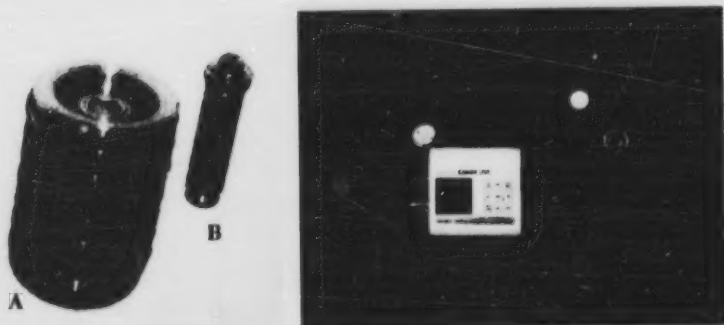


Fig. 4: Design of the Expansion-Ring-Anode (A) and cathode-bar (B) and photograph of the Expansion-Ring-System installed into a bridge cap in Germany during measurement with removed protection cap (C)

The distance between anode and cathode should be in the range of 15 cm. Additionally the electrolytic resistance between the rings can be measured to check the electrical contact between the rings and the concrete.

In the head of the Expansion-Ring-Anode a socket is integrated, which allows to insert a plug of the measuring instrument. Between the measurements a sealing cap is put onto the socket to protect it from corrosion. Cables are lead from the measuring rings to this integrated socket inside of the Expansion-Ring-Anode. After production of the electrode all inner open spaces are filled with resin and sealing material to insure that no water or chlorides can penetrate to the inner part of the sensor.

Laboratory tests have shown that the formation of rust at the first anodes after depassivation has an additional sealing effect. Therefore, the depth of the critical chloride front is not influenced by the presence of the sensors, because chlorides can only enter into the concrete besides the Expansion-Ring-Anode through the existing concrete.

4 Measurements and evaluation

To detect the onset of corrosion of the anode-rings the electrical currents, voltages, electrical resistances and temperatures of the embedded sensors are measured using a specially developed hand instrument e. g. one to four times a year.

As long as the critical chloride content and the carbonation depth have not reached the surface of the outer anode-ring, all electrical currents between the anode-rings and the cathode-bar are low. In the course of time the anodes will be depassivated one after the other. By measuring the electrical currents and voltages between anode-rings and cathode-bar in regular intervals, the critical depth regarding reinforcement corrosion is measurable continuously.

To be able to estimate additionally the potential corrosion behaviour of the reinforcement after depassivation, the electrical AC-resistance-distribution of the

concrete between the six single anodes is also measured. Generally the maximum possible corrosion rates decrease with increasing resistance of the concrete. Additionally the temperature is measured using an integrated PT 1000 sensor.

For the measurements an automatic portable instrument CANIN-LTM has been developed. Using this battery-powered instrument the electrical currents, potentials, electrolytic resistances and the temperature are measured automatically after pressing the start-button. The critical values are indicated simultaneously within the large display. The data of the measurements of 1.000 sensors can be stored and transferred to a personal-computer by the integrated serial port. For further evaluation of the data software is available, which builds up a database and shows the data versus time including alarm-values (see section 6).

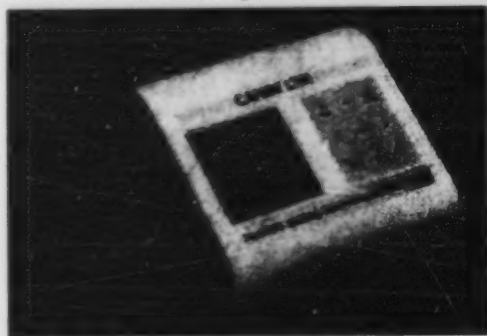


Fig. 5: Measuring Instrument CANIN-LTM

The CANIN-LTM is also used to check the proper contact of the anode-rings during installation. From the first experience it is known, that the electrical resistances between the anode-rings have to be in the range between 1 and 10 k Ω .

6 Results of laboratory tests

To investigate the correct functioning of the Expansion-Ring-System comparative tests have been carried out using concrete specimen with embedded Anode-Ladder according to [6]. Expansion-Ring-Anode and cathode have been installed 137 days after placement of the concrete. At this stage the first two anodes of the Anode-Ladder indicated activity showing that the critical depth related to reinforcement corrosion was between 2 and 3 cm as the single anodes have been installed in 1 cm - steps. To achieve this situation a chloride solution containing 1 % Cl⁻ had been applied onto the concrete surface beginning from 1 week after placement of the concrete. After installation water has been applied onto the concrete surfaces in regular intervals (see Figs. 6 and 7) to investigate the influence of wetting and drying on the measured data.

Figs. 6 and 7 show the standard plots produced by the MACRO-software developed for the monitoring systems. To get a direct overview of all data, the time-dependant values of voltage of all anodes against the cathode, electrical currents 5 seconds after coupling of the anodes against the cathode, the AC-

resistance at 100 Hz between neighboured anodes and temperature of the integrated PT 1000 sensor are plotted on 1 page. As limit values for activity of the single anodes under usual conditions 150 mV and 15 μ A after a short circuit time of 5 seconds are used from experience. For special conditions regarding concrete composition or environment, e.g. for permanently submerged concrete, other limit values might be relevant, which can be determined easily by calibration tests using the same concrete under the same conditions.

Fig. 6 and 7 show the data taken from both sensor systems beginning at the time of installation of the Expansion-Ring-Anodes during cyclic application of the 1 % Cl⁻ solution, which can especially be seen at the data of the outer anodes near the exposed concrete surface. Voltages and currents of the Anode-Ladder show that the 2 outer anodes 1 and 2 are depassivated, while the remaining anodes 3-6 are passive (Fig. 6).

Fig. 7 shows that it takes about 1 month until voltages and currents of the single anodes of the Expansion-Ring-Anode above and below the critical depth separate clearly from each other. This can be explained by the fact that it takes a certain time until the surfaces of the rings passivate due to the alkalinity of the surrounding concrete and until the chlorides cause an activation. After this period the further ingress of chlorides can also be monitored.

The differences in the absolute values for the electrical currents and resistances are caused by the different geometry of both types of sensors. For these comparative tests an Expansion-Ring-Anode with a diameter of 30 mm has been used.

This preliminary test shows that the Expansion-Ring-System is suitable to monitor the critical depths related to corrosion of the reinforcement. The long-term-tests are still running and will be presented later.

7 Fields of application

The corrosion monitoring system has the following advantages compared to other monitoring methods:

- It is not only shown, whether the reinforcement corrodes or not, but it can also be estimated, when the reinforcement will start to corrode.
- The corrosion monitoring system shows directly the depth of the critical chloride content and not the absolute chloride contents. Compared to the measurement of chloride profiles this is a decisive advantage, because as known from experience the interpretation of absolute chloride contents is often difficult. Therefore the installation of corrosion monitoring sensors can not be replaced only by taking concrete samples for chloride profiles.
- The use of the corrosion monitoring system is especially economic in not accessible locations, e. g. at the outer surfaces of tunnels or in the tidal zones of pier shafts, etc..

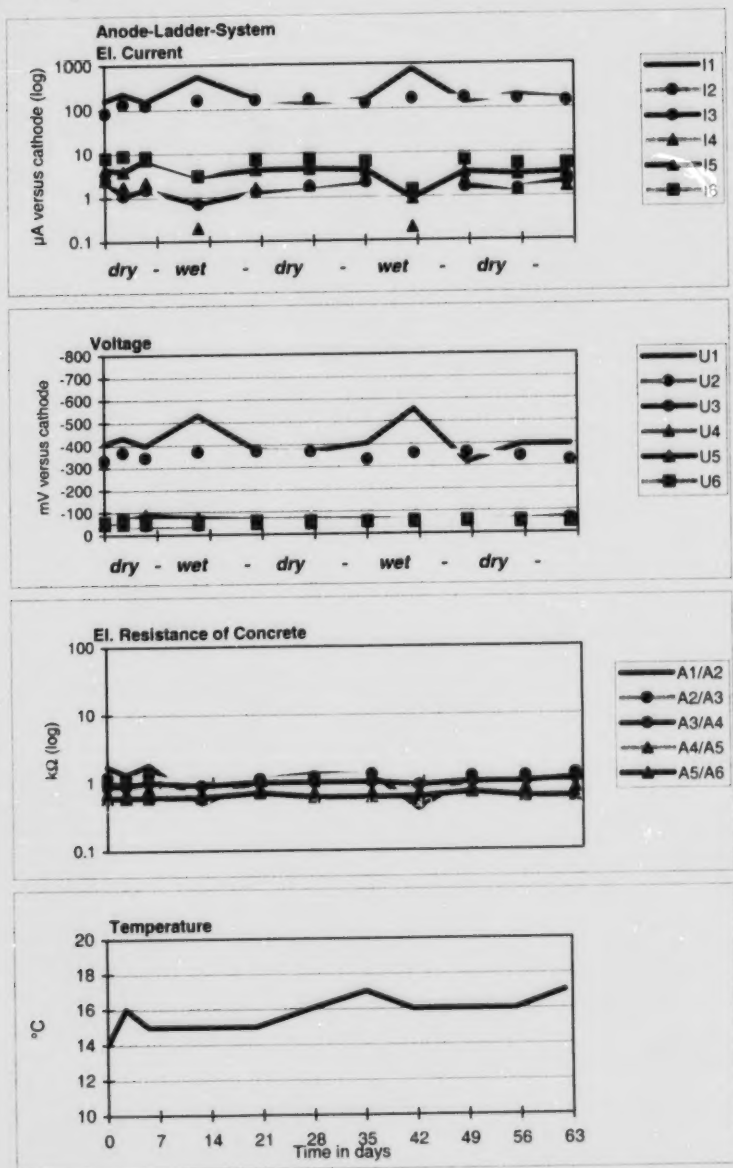


Fig. 6: Results from a laboratory test of the Anode-Ladder-System under cyclic water application (A1 is the single anode next to the concrete surface, ect)

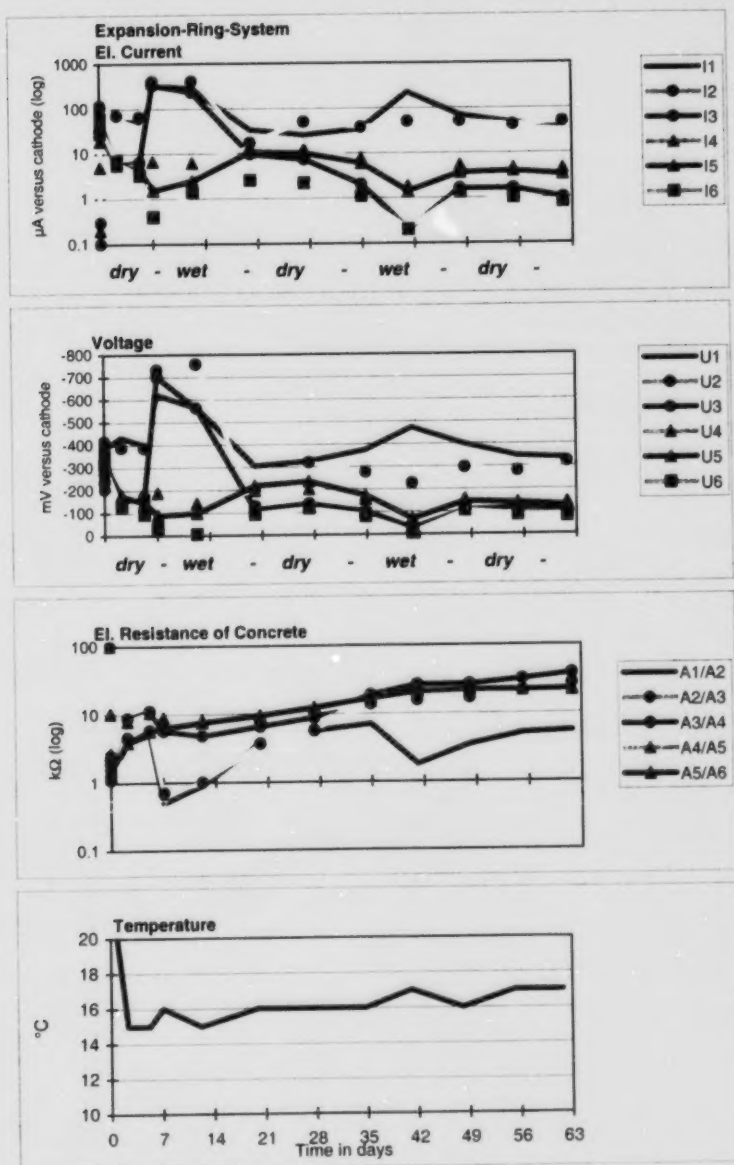


Fig. 7: Results of the Expansion-Ring-System installed near the Anode-Ladder; compare to Fig. 6 (A1 is the single anode next to the concrete surface, etc.)

When the corrosion monitoring systems are used as an effective integrated part of the maintenance program, the operating costs i.e. the inspection, maintenance and repair costs, can be reduced significantly. Possible fields of application are the following:

- Reinforced concrete structures exposed to aggressive environments (offshore structures, buildings near the coast, bridges, parking structures, tunnels, foundations, etc)
- Areas with difficult or without access (outer areas of tunnels and pipes, foundations and tanks, piers, piles of bridges near the water level, etc.)
- Monitoring of the durability of special protection systems for new structures especially with a high designed service life, e.g. 100 years (high performance concrete, coatings, inhibitors, etc.)
- Monitoring of the durability of special repair systems (cathodic protection, desalination, coatings, inhibitors, etc.)

8 Outlook

Laboratory tests and first installations on site have shown, that it is possible to monitor the critical depth related to reinforcement corrosion using the Expansion-Ring-System.

Since September 1998 the Brite Euram Project "Smart Structures" is running to develop an integrated monitoring system for concrete structures. One task of this project is the further development of the Expansion-Ring-System, especially optimisation of the installation procedure and testing in different concretes in laboratory and in critical locations of bridges. The further results will be presented within the next years

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36 A SIMPLIFIED METHOD TO ESTIMATE CORROSION RATES — A NEW APPROACH BASED ON INVESTIGATIONS OF MACROCELLS

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Abstract

A numerical simulation of the corrosion process of steel in concrete based on equivalent electrical circuit models is presented allowing for a quantitative prediction of macrocell action in reinforced concrete. The analysis has been performed for a well-defined geometry with a planparallel and a coplanar arrangement of actively corroding and passive steel. For the planparallel arrangement the conjoint action of microcells and macrocells was investigated for a two-dimensional situation. Furthermore, pitting corrosion on a single rebar was studied for the coplanar arrangement using a transmission line model. The comparison between model-tests and calculations show, that this new numerical model is suitable to estimate corrosion rates of steel in concrete.

Keywords: Steel, Reinforcement, concrete, durability, chloride, corrosion, anode, cathode, macrocell, numerical simulation

1 Introduction

Corrosion of steel in concrete has caused severe durability problems especially at concrete structures exposed to aggressive environments. The repair measures have grown to such an extent, that the operation and repair costs constitute a major part of the current infrastructure.

For durable design and repair of concrete, a basic knowledge of the underlying corrosion mechanisms is needed. Especially the estimation of the corrosion rates of the rebars under different conditions plays an important role.

This paper describes a simplified model based on the electrochemical nature of the corrosion process in order to calculate corrosion rates under well defined assumptions. The model is based on the results from the extensive experimental research of both authors on corrosion of steel in concrete, especially on macrocell action, which has been carried out during the last 10 years.

2 Description of the corrosion process

The mechanism of steel corrosion in concrete is essentially electrochemical in nature, i.e. the corrosion process proceeds by forming spatially separated anodic and cathodic areas on the surface. According to this view a corroding steel surface may be regarded as a composite of anodic and cathodic areas electrically coupled through the body of the steel itself to allow movement of electrons, and electrolytic movement of ions through the electrolyte solution contained within the concrete pore network to permit electromigration and diffusion of ions. With respect to the spatial distribution of anodic and cathodic sites over the steel surface, a distinction has to be made between micro- and macro-corrosion cells [1,2,3].

Uniform corrosion of the reinforcing steel is normally associated with the existence of microcells. These electrochemical cells are characterised by a completely random distribution of anodic and cathodic sites interspersed all over the surface, owing to local chemical or physical differences in or on the steel.

The different exposure conditions throughout the structure and local differences in the concrete properties will undoubtedly promote the development of so-called macrocells. Many factors can set up potential differences between adjacent parts of the steel reinforcement network. Common examples for corrosion of the reinforcement include differential aeration cells and active/passive cells.

The corrosion process induced by the presence of chloride ions at the steel-concrete interface is normally associated with the development of active-passive cells. The local active steel surface predominantly acts as an anodic site whereas the adjacent steel remains passive and consequently acts as a cathodic area. The resulting galvanic action between active and passive steel may have a pronounced effect on the corrosion rate at the anodic site. A macroscopic active/passive corrosion cell can possess both a large driving voltage and a large-cathode/small-anode ratio known to be most detrimental. Under these conditions substantial, strongly localized reductions in the cross sectional area can occur (pitting).

3 Simplified calculation model for macrocells

The electrochemical behaviour of a simple macrocell involving one anodic and one cathodic component can be evaluated by using an appropriate equivalent electrical circuit. A two-component macrocell is considered to comprise a separate anodic and cathodic resistance against polarisation, $R_{p,a}$ and $R_{p,c}$ respectively, linked by an electrolytic concrete resistance, R_{con} , in series. The anodic and cathodic resistances reflect the polarisation behaviour of the steel sections involved. Both interface resistances exhibit a pronounced non-linear current-voltage relationship.

Macrocell action between actively corroding and passive steel can thus be expressed mathematically by the following equation:

$$I_{gal} = \frac{U_c - U_a}{R_{p,c} + R_{con} + R_{p,a}} \quad (1)$$

The magnitude of the galvanic current, I_{gal} , flowing between the anodic component (actively corroding steel) and the macro-cathode (passive steel) is

determined by the ratio of the potential difference between passive (cathodic) and corroding (anodic) areas, $(U_C - U_A)$, and the total resistance of the galvanic circuit. This basic relationship permits the numerical simulation of macrocells for both planparallel and coplanar arrangements of reinforcement steel.

Actively corroding steel may exhibit microcell action in which anodic and cathodic sites are randomly distributed over the exposure surface of the steel electrode giving rise to a uniform corrosion attack. In this condition the polarisation behaviour of corroding steel is described by the following equation relating average current density, i , and potential change (overvoltage), ΔU :

$$i = i_{\text{corr}} \left[\exp \left(\frac{\ln(10) \Delta U}{b_a} \right) - \exp \left(\frac{-\ln(10) \Delta U}{b_c} \right) \right] \quad (2)$$

with $\ln(10) = 2.303$

For the anodic and cathodic Tafel slopes, b_a and b_c , values of 90.7 mV/dec and 176.3 mV/dec, respectively, are introduced which results in a Stern-Geary constant $B = 26.0$ mV [4]. At the free corrosion potential, E_{corr} , i.e. without any macrocell effects, the steel corrodes uniformly with a corrosion current density of i_{corr} [$\mu\text{A}/\text{cm}^2$]. For the planparallel electrode arrangement, microcell activity is considered for the anodic component with $i_{\text{corr}} = 1.0 \mu\text{A}/\text{cm}^2$. If small concentrated anodes are considered, i.e. for the coplanar arrangement, it is assumed that the local anode exhibits no significant resistance against polarisation. In the latter case the macrocell current is equal to the corrosion current of the local anode.

For passive steel it is assumed that anodic reactions can only proceed to a very limited extent. The electrochemical behaviour of passive reinforcing steel under cathodic polarisation is given by:

$$i = \frac{1 - \exp \left(\frac{-\ln(10) \Delta U}{b_c} \right)}{\frac{1}{i_{\text{corr}}} - \frac{\exp \left(\frac{-\ln(10) \Delta U}{b_c} \right)}{i_{\text{lim}}}} \quad (3)$$

based on data obtained from experimental investigations [2, 5].

For passive steel values of $1 \cdot 10^{15}$ mV/dec and 176.3 mV/dec for the anodic and cathodic Tafel slopes, b_a and b_c , respectively, were introduced which results in a Stern-Geary constant $B = 76.55$ mV. The corrosion current density of passive steel at the steady state potential, i_{corr} , is set at $0.01 \mu\text{A}/\text{cm}^2$ [2]. The influence of oxygen diffusion through the concrete cover on the polarisation behaviour is taken into account by introducing a limiting diffusion current density, i_{lim} [6]. It is assumed that oxygen diffusion has a significant effect for coplanar macrocells but not in the case of planparallel arrangements of reinforcing steel [5].

Fig. 1 shows the cathodic polarisation curve of passive reinforcement for different limiting currents according to equation [3].

Current of Macro-Cathode in $\mu\text{A}/\text{cm}^2$

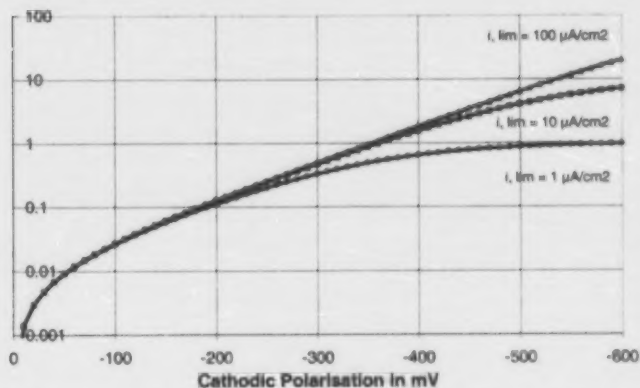


Fig. 1: Cathodic polarisation-curves of passive reinforcement for different limiting currents i, lim

4 Macrocells with planparallel arrangement of corroding steel and macro cathode

For the numerical simulation of a planparallel arrangement, a basic set-up of a macrocell is considered (see figure 2). The macrocell consists of two rebars with a diameter of 16 mm placed in parallel being separated by 200 mm of concrete. This arrangement results in a corroding to passive surface area ratio of 1.0. Both rebars have a cover depth of 32 mm; the width of the specimen amounts to 200 mm whereas a unit length of the rebars of 100 mm is considered resulting in an exposure surface of 50.27 cm^2 for each rebar.

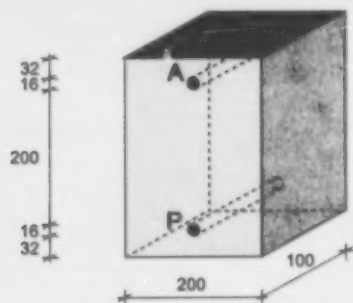


Fig. 2: Cross section of macrocell specimen with planparallel rebar arrangement

The electrochemical behaviour of the steel-concrete interface of each rebar is modelled with 16 discrete equidistant interface elements. The electrical properties of the concrete environment are modelled by pure resistive elements each representing a square cross section of $4 \times 4 \text{ mm}^2$.

As a starting point the corrosion current density for the corroding steel (denoted rebar A) due to microcell action is arbitrarily set at $i_{\text{corr}} = 1 \text{ } \mu\text{A}/\text{cm}^2$, resulting in a total corrosion current, $I_{\text{corr}} = 50.27 \text{ } \mu\text{A}$. For the macro-cathode (denoted rebar P) the corrosion current density is set at $i_{\text{corr}} = 0.01 \text{ } \mu\text{A}/\text{cm}^2$ reflecting the passive condition of the steel. The concrete resistivity, $\rho_{\text{con}} [\Omega\text{-cm}]$, and the initial potential difference between the passive and the corroding steel, $U_{\text{oc}} = (U_{\text{C}} - U_{\text{A}}) [\text{mV}]$, are varied within a wide range during the numerical simulation.

5 Corrosion current and resistance control for planparallel rebar arrangement

Due to macrocell action the kinetics of both the anodic and cathodic processes of the corroding rebar are affected. In Figure 3 the resulting change in the corrosion current density, i_{corr} , i.e. self-corrosion rate including macrocell action, is shown as a function of open-circuit voltage U_{oc} , for different practical values of concrete resistivity, ρ_{con} . For $U_{\text{oc}} \leq 400 \text{ mV}$ the influence of macrocell action is limited since it does not result in a significant acceleration of the corrosion rate. However, for $U_{\text{oc}} \geq 400 \text{ mV}$ the magnitude of i_{corr} is increased from $1.0 \text{ } \mu\text{A}/\text{cm}^2$, i.e. the initial corrosion without macrocell action, to values ranging from $1.23 \text{ } \mu\text{A}/\text{cm}^2$ (400 mV; 50000 Ωcm) to $3.52 \text{ } \mu\text{A}/\text{cm}^2$ (600 mV; 5000 Ωcm). The corresponding potential of the anodic components shifts by +8.0 to +49.5 mV (anodic polarisation) and for the macro-cathode from -268.8 to -436.7 mV (cathodic polarisation).

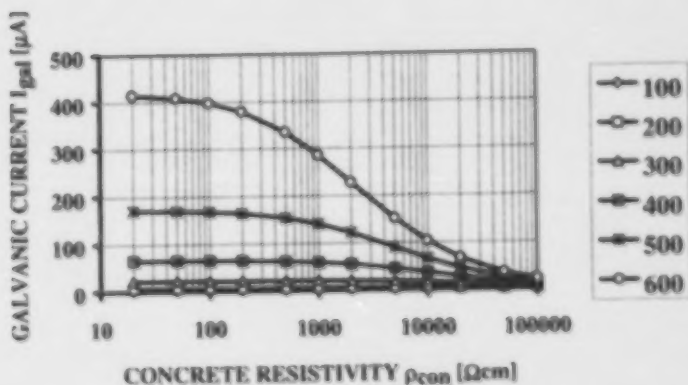


Fig. 3: Macrocell current as a function of concrete resistivity

These potential shifts of the anode and macro-cathode already indicate that the control exerted by the concrete resistance on the macrocell process is significantly lower than the control exerted by the cathodic process of the passive steel. As an example Figure 4 shows the relative resistance control of the three processes involved, i.e. the anodic, cathodic and electrolytic process, denoted C_a , C_c , C_d , respectively, as a function of concrete resistivity for $U_{oc} = 400$ mV. For the complete range of concrete resistivities investigated the cathodic control, C_c , clearly dominates starting with a value of 93% ($20 \Omega\text{cm}$) which gradually decreases to 59% for $\rho_{con} = 100000 \Omega\text{cm}$. The increase of concrete resistivity is reflected in a simultaneous rise of the control exerted by the concrete resistance, C_d , from 0% to 40%. For $U_{oc} = 600$ mV and $\rho_{con} = 5000 - 50000 \Omega\text{cm}$, the contribution of the concrete resistance to the overall macrocell circuit resistance ranges from 19% to 43% and for $U_{oc} = 400$ mV from 9% to 31%. Due to the high corrosion rate the relative resistance control of the resistance against polarisation of the actively corroding steel, C_a , is always less than 10%.

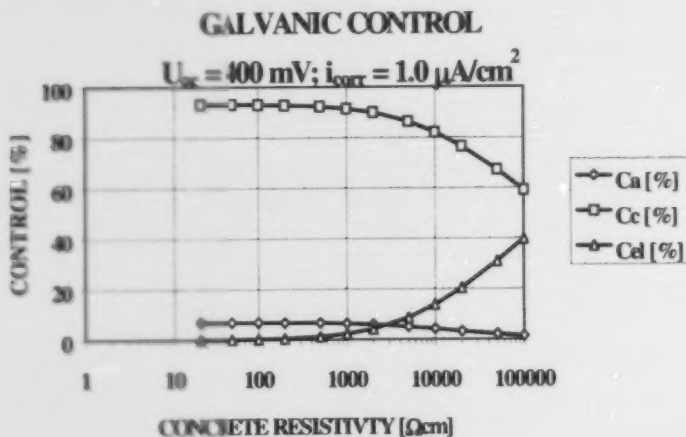


Fig. 4: Relative control of the anodic, cathodic, and electrolytic process on the overall macrocell resistance for planparallel rebar arrangement

6 Macrocells with coplanar arrangement of local anode and macro cathode

A single rebar may exhibit macrocell action when large areas of passive steel are interspersed by small anodic sites. Such a situation may develop when cracks in the concrete cover cross underlying reinforcing bars. The corrosion behaviour of this macrocell may be simulated by a one-dimensional approach using a transmission line model. The passive steel acts as macro-cathode which is subdivided into 50 discrete elements along the rebar, each with a certain potential and current. The individual cathode-elements have a standard length of 10 mm and a diameter of 16 mm.

Between neighbouring elements an ohmic drop over the concrete electrolyte

is induced by the flow of ionic current along the rebar. The calculation procedure for macrocells using a transmission model is described in more detail in [2].

Figure 5 depicts the distribution of the cathodic current over the passive steel as a function of the distance from the central corroding zone (ideal anode) for a wide range of driving voltages between corroding and passive zones, $U_{oc} = 100$ -600 mV, using the following assumptions:

- Anode: anodic polarisation resistance = 0Ω , simulating e.g. extremely high chloride contents; no cathodic reaction on the surface of the anodic component;
- Concrete: longitudinal electrolytic resistance along the rebar: $r_{con} = 100 \Omega/\text{cm}^1$;
- Macro-cathode: polarisation behaviour given by eq. [3] with $i_{lim} = 100 \mu\text{A}/\text{cm}^2$.

The first assumption implies that the corrosion current of the anode is given by the overall cathodic current produced by the passive steel area.

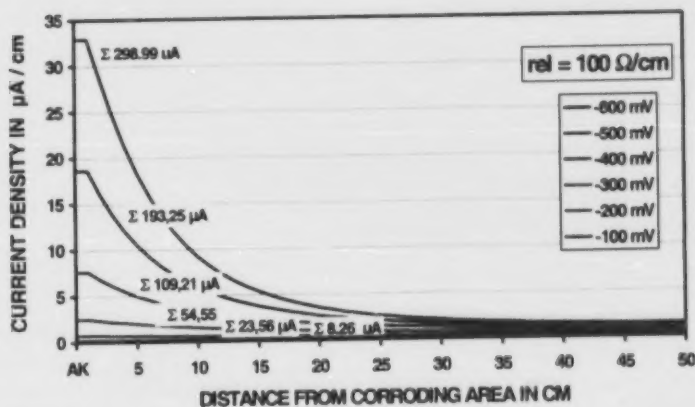


Fig. 5: Cathodic currents for a coplanar macrocell arrangement (example)

As to be expected, the cathodic current density increases considerably with higher driving voltages, and decreases with the distance from the corroding area. However, significant cathodic current densities are not only present within a few cm from the corroding zone, but also at passive steel surfaces more distant from the anode. Especially when high driving voltages are present, passive steel at a distance more than 20 cm still may contribute significantly to the overall cathodic current. The distance of influence may then even amount to several meters in case of low resistivity concrete or in cases with large distance between depassivated and well aerated areas.

6 Investigations to verify the numerical models

Two reinforced concrete beams with a length of 2 m have been produced to study the cathodic current distribution of passive reinforcement within a one-dimensional electrical field to verify the transmission line model. Altogether 20 pieces of reinforcement have been embedded each with a length of 10 cm to simulate one 2 m bar (see Fig. 6). At one side 2 % and 3 % of chlorides / cement have been added to the concrete mix while the remaining 19 segments have been produced using concrete without chloride addition. Connecting an amperemeter between the cables of the rebars it was possible to measure the macrocell currents as shown in Fig. 6. Details are described in [2].

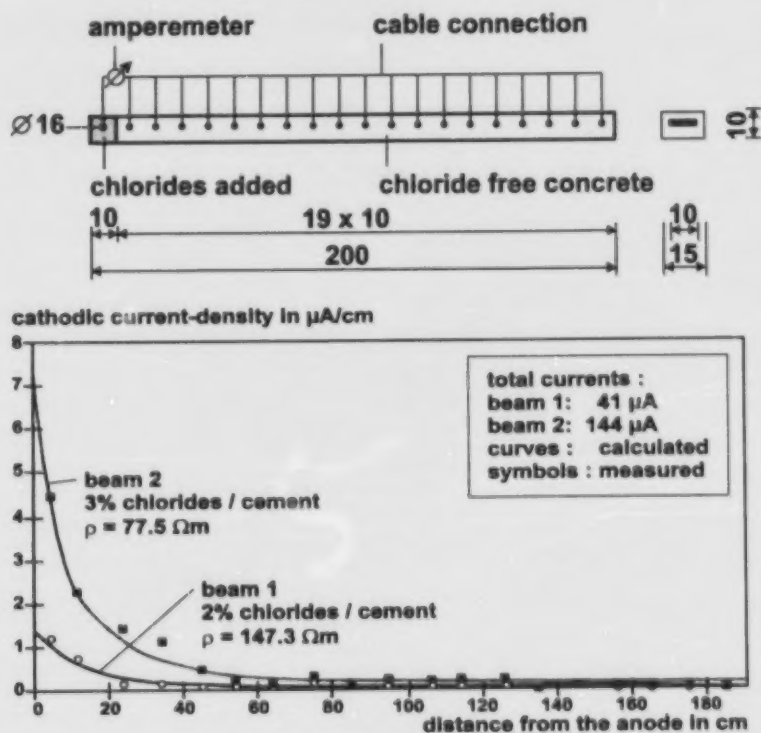


Fig. 6: Test set-up and cathodic current distribution of the passive reinforcement

The test results show a good correlation between measured values and the data calculated using the transmission line model using the formula describing the polarisation behaviour of passive steel (equation [3]). However, for geometrical conditions of macrocells occurring in practise more complex models for the description of the electrical fields have to be used for the numerical simulation.

7 Determination of steel removal rates

The basic relationship between anodic current and steel removal rate is given by Faraday's law. In Fig. 7 the resulting removal rate [$\mu\text{m/a}$] due to macrocell action, i.e. due to the current supplied by the macro-cathode, is shown for various anode surface areas, assuming uniform corrosion attack over the exposure surface. In the case of non-uniform corrosion attack, the surface areas showing the highest local corrosion rates are considered of most practical relevance. Therefore an average value for the anode surface may be used as an input for Fig. 6.

For planparallel macrocells the steel removal rates can be approximated as the sum of the initial corrosion current generated by self corrosion and the macrocell current supplied by the macro-cathode. Generally, the macrocell current underestimates the actual rate of attack, since cathodic reactions on the corroding rebar are not completely suppressed.

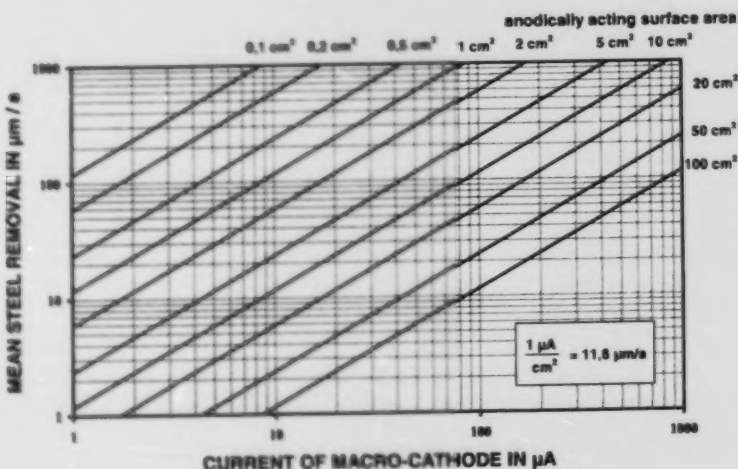


Fig. 7: Relationship between cathodic current and steel removal rate

For the coplanar arrangement the overall cathodic current is equal to the corrosion current of the anode. Hence, the magnitude of the cathodic current can be converted directly into the rate of attack.

8 Concluding remarks

Macrocell action plays an important role in the corrosion process of reinforcement steel. For practical situations this results in complex electrochemical interactions involving all parts of the steel reinforcement network. These effects can be quantified by a numerical analysis which permits a prediction of the current distribution for arbitrary reinforcing steel geometry's,

electrochemical characteristics of the steel/concrete interface, and concrete resistivity distributions [5,7].

The magnitude of the macrocell current flowing between the macro-cathode and the anodic component is strongly dependent on the driving voltage, the resistivity of the concrete electrolyte, and the polarisation behaviour of the passive steel. Therefore simplified models based simply on the influence of the electrolytic resistivity on the concrete might be misleading in many cases.

Generally, the rate of attack is determined by the conjoint action of microcells and macrocells. Under adverse conditions the local corrosion rate may be multiplied compared to the initial situation without macrocell action. The situation when the macrocell current equals the corrosion current is approached when the overall anodes of the macrocell system are polarised in the anodic direction by more than approximately 50 mV from their respective initial free corrosion potential.

In general, the highest corrosion rates have to be expected when the corroding area is small compared to the exposure surface of the macro-cathodes. The situation is exacerbated when the concrete has a low resistivity and a high permeability for oxygen. However, restriction of the corrosion rate by oxygen diffusion is not to be expected for most practical situations unless the concrete cover is water saturated during an extended period of time [6].

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37 CHARACTERISING CHLORIDE PENETRATION RESISTANCE OF CONCRETE

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Abstract

Concrete in coastal and marine environments can be subjected to a number of deterioration mechanisms. They include carbonation and/or chloride-induced steel corrosion and sulphate attack. Recent emphasis has been given to the premature deterioration caused by steel corrosion. Performance requirements are generally based on factors that reflect the resistance of concrete to chloride penetration. The interactions between different deterioration mechanisms are discussed. In most cases, chloride-induced corrosion is believed to be the prime deterioration mechanism. This is because conditions, that favour a rapid carbonation and chloride penetration, rarely coexist. The expansion of concrete by sulphate ions in seawater is retarded owing to the concomitant presence of chlorides. A number of established and emerging methods used in comparing the resistance of concrete to chloride penetration are discussed. It has been found that there are great limitations on the suitability of a range of indirect methods used. The merits of assessing the chloride penetration profile, diffusion coefficient, and accelerated methods based on the ASTM C1202 and its modification are presented. Australian data presented clearly highlight the benefits of the use of a range of supplementary cementing materials (SCM) in improving the resistance of concretes to chloride penetration.

Keywords: Chloride penetration, carbonation, sulphate attack, diffusion, sorptivity, total charge passed (ASTM C1202), supplementary cementing material (SCM)

1 Introduction

All major urban developments in Australia occur along its extensive coastline. With concrete being the most commonly used construction materials, the performance of concrete in marine environments is critical to the durability of a great proportion of infrastructures in Australia. Chloride-induced corrosion is considered to be the main deterioration mechanisms affecting the durability of concrete structures in coastal and marine environments. This is so despite the fact that both carbonation and sulphate attack can each coexist with the chloride-related deterioration.

Corrosion is initiated when there is damage to the passivated layer of steel reinforcement due to the availability of sufficient chloride ions at the steel/concrete interface; and/or there is a loss of alkalinity of surrounding concrete due to reaction of carbon dioxide with various hydrates in the hardened concrete (carbonation).

1.1 Chloride-carbonation interactions

The coexistence of carbonation and chloride penetration in concrete is generally rare in practice. However, their combined effect could lead to a rapid deterioration. Earlier work by Roper and Baweja (1991) and subsequent research at CSIRO found that the presence of chloride ions in carbonated concrete resulted in very high corrosion rate of the embedded steel reinforcement. The corrosion rates were of an order of magnitude higher than for carbonated concrete in the presence of water. In a number of field investigations reported by Sirivivatnanon et al. (1996) and Driscoll et al. (1997), no significant carbonation depths were found in concretes exposed to the atmospheric, tidal or submerged pool zones for marine structures. Current investigation of a salt water swimming pool in Wollongong reveals carbonation in parts of the concrete columns. These columns are part of the grandstand structure of the pool. They showed a carbonation depth of 10 mm after 60 years of exposure to the atmosphere at this beachfront location. A concrete cover much greater than 10 mm is usually specified for concrete exposed to such an environment. It is well established that the rate of carbonation is slow at extreme relative humidity conditions. Concrete exposed to marine conditions is usually subjected to a high relative humidity and hence a very low carbonation rate is expected. Carbonation/chloride interactions are therefore expected to be applicable to limited and very special circumstances. It is therefore not surprising that chloride penetration has been the main focus of engineers' attention in dealing with steel corrosion of structures in coastal and marine environments.

1.2 Chloride sulphate interactions

In the past, sulphate resistance cement was commonly specified for concrete in marine structures. Nowadays, it is generally believed that deterioration of concrete matrix due to sulphate attack occurs at a much lower rate than the adverse effect from steel corrosion.

Mehta (1986) gave a comprehensive review of physical and chemical processes involved in the sulphate attack which manifests itself in the form of expansion-cracking, or progressive loss of strength, or both. In the physical processes, the

permeability of concrete plays a very important role in determining the severity of sulphate attack. The attack is limited to the surface and is usually of no consequence when the concrete has low permeability. Lea (1970) and Biczok (1980) confirmed that the expansion of concrete by sulphate ions in seawater is retarded owing to the concomitant presence of chlorides. The rate of diffusion of chloride ions, being much higher than that of the sulphate ions, allow the chloride ions to permeate through the concrete surface much faster than the sulphate ions. Chlorides permeating into the concrete react with C_3A to form calcium chloroaluminate hydrate (Friedel's salt), and, as a result, the quantity of C_3A available for the sulphate ions to react and form either gypsum or expansive ettringite is reduced. Al-Amoudi et al. (1995) confirmed that the presence of chloride ions in the sulphate environment mitigated the sulphate attack in plain and blended cements.

2 Quantifying chloride penetration resistance

There are a number of approaches to quantify the chloride penetration resistance of concrete. They include the determination of chloride penetration after exposure in a chloride solution or seawater. The concrete used can be samples prepared in a laboratory mix or cores taken from the field. Numerous types of exposure conditions have been used including a simple ponding (partial- or full-immersion) or a simulated tidal exposure (wetting and drying) or actual field exposures. The mediums used are either sodium chloride solutions of different concentrations, synthetic ocean water or the actual medium in the field. There are also accelerated methods that may involve the use of a number of mediums and accelerating regimes.

2.1 Direct method of assessment

The most commonly used methods of assessment are:

- determination of *chloride front* with silver nitrate indicator,
- *chloride penetration profile* from analysis of chloride ions of samples taken at different depths from the exposed surface, and
- migration tests involving the measurement of chloride level of the solution in the receiving chamber opposite the source chamber.

The silver nitrate indicator appears to be the simplest method for determination of the depth of chloride penetration into concrete. It has been reported by Otsuki et al. (1992) that by spraying a 0.1 N $AgNO_3$ solution on freshly broken concrete sample, the presence of chloride, from a minimum concentration at 0.15% by mass of cement, can be detected by the change in the colour of the sprayed surface.

Determination of chloride profile is possibly the most popular tool used to assess the chloride resistance of concrete both qualitatively and quantitatively. In qualitative terms, comparisons are made after a similar exposure period of various concrete samples. If it is assumed that diffusion is the main transportation mechanism of chloride into the concrete. Then according to Fick's second law, a diffusion coefficient can be calculated by fitting an error function to the measured chloride profile. It must be noted however that a chloride diffusion coefficient, determined from a measured chloride profile, is not unique in value. The coefficient is termed an 'apparent chloride diffusion coefficient - D_a '. It can vary significantly with the periods of exposure. This will be further discussed.

Many researchers have successfully developed 'migration' or 'concentration difference diffusion' testing techniques to measure the steady-state chloride diffusivity of concrete. The chloride concentration gradient, obtained from the measured concentrations of the source and receiving chambers, enables the calculation of diffusion coefficient. Some of these techniques are described by Dhire et al. (1990) and Gjrv (1996). Andrade and Sanjun (1994) outlined an experimental procedure for the calculation of chloride diffusion coefficients in concrete from migration tests. They based on the theoretical approach using the Nernst-Planck equation which models mass transport in electrolytes and the Nernst-Einstein equation using electrolyte equivalent conductivity as the main parameter.

2.2 Indirect method of assessment

A number of indirect and accelerated methods have also been used to indicate the performance of concrete in marine exposures. They are:

- water permeability (German Standard DIN 1048 Part 5),
- compressive strength and water-to-binder ratio (Australian Standard AS 3600),
- Rapid chloride permeability (AASHTO 277-83) or 'Electrical indication of concrete's ability to resist chloride ion penetration' (ASTM C1202) or 'conductivity' or 'charge passed'

In Australia, the standard for concrete structures AS 3600 uses a range of strength grades and cover thicknesses as key requirements for durability. The commentary to AS 3600 also recommends a maximum water-to-cement ratio for each strength grade. In other specifications, a mix of prescriptive and performance-based requirements is being introduced in the last few years. Accelerated electrical techniques such as the measurement of total charge passed according to ASTM C 1202 have also been used. The relevance of ASTM C 1202 method to the resistance of concrete to chloride penetration has recently been reviewed by Cao et al. (1996). Water permeability is also strongly advocated by a number of researchers. In terms of the performance-based specification of HPC, it has been found by Sirivivatnanon and Cao (1998)¹⁷ that the total charge passed method, such as the ASTM C1202 or the CSIRO-modified method, is one most promising method for coastal and marine structures.

3 Characterising chloride penetration resistance

The resistance to chloride-induced corrosion largely controls the durability performance of most reinforced concrete in coastal and marine environment. It has also been widely found that those parts of the structure exposed to the tidal and splash zone are the most vulnerable. Chloride-induced corrosion mechanism can be represented by a simple model of initiation (chloride penetration and depassivation of steel) and propagation. Many engineers have associated the service life mainly with the initiation stage as it represents a time span before costly procedures are required to lengthen the service life of a concrete structure.

In gauging the initiation period, the resistance of concrete to chloride penetration and the chloride threshold for depassivation of steel reinforcement are both important. It is the first of these two factors, which has been widely used as an indicator of the relative performance of concrete. There has been no consensus on the

chloride threshold for depassivation. Earlier CSIRO work reported by Cao et al. (1988) found the threshold to vary depending on the type of cement and curing. The findings contradicted claims that a lower threshold was expected for blended cements due to reduction of the OH^- concentration.

3.1 Diffusion of non-ionic solution

Fick's first law of diffusion or steady state diffusion law

$$J = -D_e \frac{\delta C}{\delta x} \quad (1)$$

Measurement of the diffusion coefficient involves the concentration difference at any two particular planes at x and $x+dx$. The coefficient represents the net chloride flow through concrete. It is referred to as an effective diffusion coefficient D_e .

Fick's second law of diffusion or transient state diffusion law is given as:

$$\frac{\delta C_1(x,t)}{\delta t} = -D_a \frac{\delta^2 C_1(x,t)}{\delta x^2} \quad (2)$$

where C_1 denotes the total concentration of chloride ions, and D_a assumed constant. The solution for a semi-infinite medium with initial conditions of $C_1(x>0,0)=0$ and boundary condition $C_1(0,t)=C_s = \text{constant}$ is described by:

$$C_1(x,t)/C_s = \text{erfc}\left(x/2\sqrt{D_a \times t}\right) \quad (3)$$

where erfc denotes the complementary error function. This solution is applicable for the unsteady flow condition where the fluid properties at any point may change with time.

3.2 Ionic diffusion

The equations (1) to (3) are valid only for an undissociated diffusant. In an ionic solution such as chloride solution, they do not strictly apply. It has been shown by Chatterji (1994a, 1994b) that Fick's laws of diffusion need modifications before they could be applied to cement based materials.

3.3 An engineering solution

In practice, an apparent diffusion coefficient, calculated from equation (3), is commonly used to characterise a particular chloride profile. Three approximations are assumed. Firstly the chloride profile of the concrete examined is assumed to be obtained from an unsteady state conditions. Secondly the ionic nature of the chloride in seawater is ignored and treated as undissociated molecules. Thirdly, the chloride profile is often based on the determination of water-soluble chloride. D_a obtained with this approximation can be adequately used to characterise the profile but cannot be directly used to predict service life.

The total concentration is measured as acid-soluble chloride ions. D_a therefore takes into account the chemical reactions within the concrete. For example, chloride ions may react with the C_3A to form calcium chloro-aluminates. The reaction or adsorption of chlorides by cement pastes is considered by Andrade (1993) to be of minor influence although a number of researchers such as Tritthart (1989) and Hachani et al. (1991) do take it into account. The free chloride ion, measured as water-soluble chloride, is however recognised to be responsible for the degradation of concrete. The use of water-soluble chloride as total chloride is therefore a convenient approximation adopted for much of the work by the CSIRO.

3.4 Dependency of D_a to exposure periods and maturity of concrete

In gauging the appropriate level of D_a for High Performance concrete (HPC), it must be recognised that in practice D_a is not unique for particular concrete. Firstly, D_a does vary with the maturity of the concrete. Secondly, for concrete of constant maturity, D_a is also dependent on the time of exposure to a chloride solution. This is possibly because of the application of diffusion measurement of non-ionic solution to an ionic diffusion.

Gjorv et al. (1994) found the apparent diffusivity obtained from three years exposure of concrete samples to seawater to be significantly lower than (about one half to one third) the same obtained from specimens exposed for one year. Dhir et al. (1991) demonstrated that steady-state conditions were not achieved during laboratory penetration tests. The apparent diffusion coefficients obtained from different exposure periods of a range of concretes were shown to reduce with increasing exposure periods. They also found that diffusion coefficients D obtained from a concentration difference (CD) diffusion test of the same concrete were consistently lower than all the D_a and represented the steady-state condition.

In recent field investigations, reported by Sirivivatnanon et al. (1996) and Driscoll et al. (1997), of the resistance of concretes against chloride penetration up to 30 years, uncontaminated and mature samples were taken and exposed to 3% NaCl solution for different periods. The results revealed the dependence of D_a to the exposure periods as shown in Table 1 and 2. Longer exposure results in a reduction of the apparent chloride diffusion coefficient for concrete of the same maturity (25-30 years). The extremely low D_a found from chloride profiles of the in-situ concrete cover is the result of the very long exposure (25-30 years) as well as the continued hydration of the concrete. It is therefore important that D_a derived from a long and similar length of exposure period is used in comparing the performance of different concretes.

Table 1: Apparent chloride diffusion coefficients of Munmorah fly ash (FA) and portland cement (PC) concretes. Sirivivatnanon et al (1996)

Type of Concrete	Location (date of sampling)	Apparent Chloride Diffusion Coefficient ($10^{-12} \text{ m}^2/\text{s}$) of 30 years old concrete after different exposure periods		
		28 days	100 days	30 years
FA concrete	M (Mon 12)	10.0	5.95	0.314
	W (Wed 14)	13.2	12.8	1.05
PC concrete	F (Fri 16)	25.4	8.6	0.232
	T (Tue 20)	39.6	27.4	0.194

Table 2: Apparent chloride diffusion coefficients of Iluka concretes. Driscoll et al (1997)

Type of Concrete	Location	Apparent Chloride Diffusion Coefficient ($10^{-12} \text{ m}^2/\text{s}$) of 25 years old concrete after different exposure periods		
		28 day	100 day	25 year
Portland cement concrete	Structural Pile No. 1	6.88	-	1.30
	No. 2	12.2	-	1.29
	No. 4	4.36	-	1.14
	Average	7.81	-	1.24

4 Binder affects chloride penetration resistance

In a major experimental program conducted at the CSIRO, a range of structural concretes was designed from four common binders and a water reducing agent. These binders are designated as follows:

- GP a General Purpose cement
- FA a 30 wt.% fly ash blend
- SF a 7 wt.% silica fume blend
- HS a 65 wt.% ground granulated blast furnace slag blended cement

Concrete of nominal strength grades of 20, 40 and 50 were manufactured in a laboratory to give a slump of 75 ± 25 mm. Standard 100-mm diameter cylinders were wet cured for 28 days for compressive strength determination. Other samples were subjected to 7 days of sealed curing in plastic sheet and subsequently stored in a standard laboratory condition until the age of testing. Circular disc samples were prepared from standard 100 mm diameter cylinders for the determination of total charge passed according to ASTM C 1202.

A set of 100 mm diameter x 50 mm height discs, epoxy coated on all except one exposed circular surface, were fully immersed in 3% NaCl solution at the age of 28 days for a full immersion period of one year before incremental drilling. The powder samples were analysed for water soluble chloride ion content by ion chromatography.

4.1 Binder independence

Because of the variation of D_a with exposure periods, values obtained from a longer exposure period, in this case for one year, are preferred in gauging the likely long-term performance of the concretes as shown in Figures 1-3.

For each binder, the influence of mix design, as reflected in the compressive strength and water-to-binder ratio, on the resistance to chloride penetration of concrete is evident. Higher strength or lower water-to-binder ratio resulted in better concrete. For concretes with the same strength or water-to-binder ratio or total charge passed, it is quite clear that D_a is highly dependent on the type of binder.

In considering the use of compressive strength, water-to-binder ratio and total charge passed as performance indicator for chloride resistance, the binder dependency of each of these properties should be closely examined. As compressive strength is a measure of the physical resistance of concrete to external force, no improvement

can be foreseen. Some improvement can possibly be made to the water-to-binder ratio by the introduction of equivalent cementing factors for different supplementary cementing materials. The total charge passed (ASTM C 1202) method can be further improved as reported by Cao et al.(1996). The modification involved performing an additional test using curing water as electrolyte. The difference in the total charges passed obtained between the standard test and test performed in water is used as an indicator for concrete quality. By using the modified method, improvements in the relationship to chloride penetration were evident.

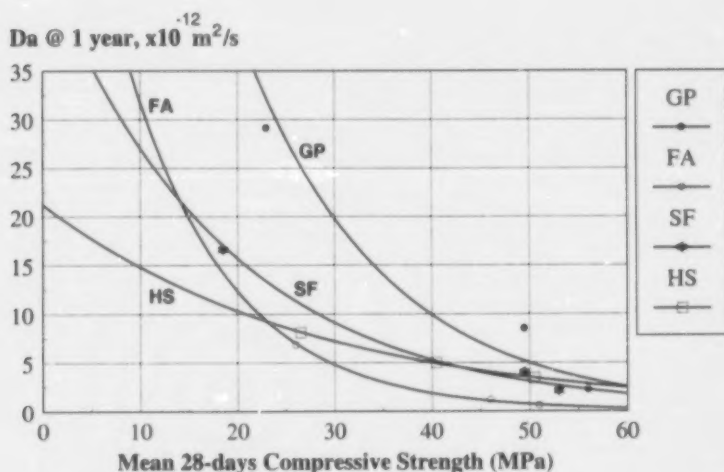


Fig. 1: Effectiveness of strength as a performance indicator

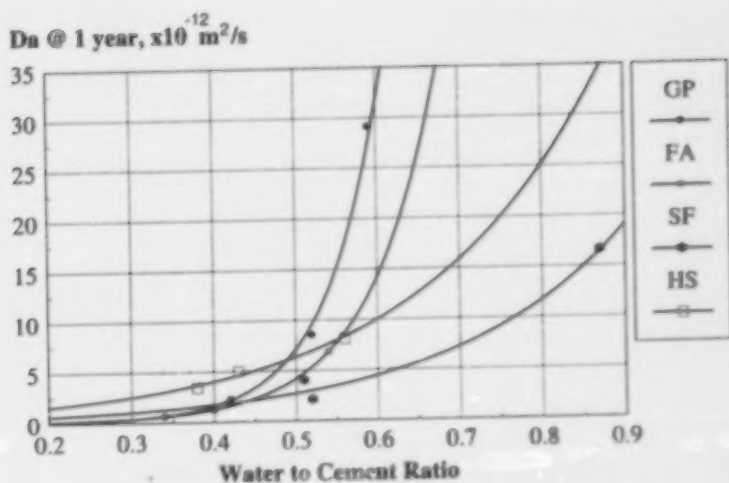


Fig. 2: Effectiveness of w/c ratio as a performance indicator

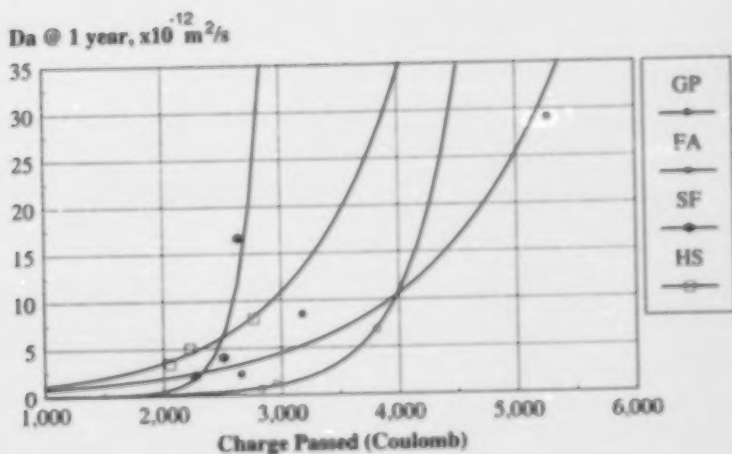


Fig. 3: Effectiveness of total charge passed (ASTM C1202) as a performance indicator

5 Discussion

The major mechanisms of deterioration of concrete in coastal and marine environments and their interaction have been examined. Chloride-induced corrosion has been singled out as the prime deterioration mechanism. In some special circumstances, carbonation/chloride or sulphate attack could be more prevalent. One possible scenario is the use of concrete culverts in arid areas of Australia where carbonation rate can be quite high. Subsequent rise of the saline water from the soil could result in severe chloride-induced corrosion of the reinforcing steel. Recognition of these possible mechanisms is important in the prevention and solving of premature deterioration.

A range of direct and indirect methods is being used to quantify the chloride penetration resistance of concrete. Chloride profile and the charge passed (ASTM C1202) are the most popular direct and indirect methods respectively. This is possibly due to their practicality, which outweighs their shortfalls. An examination of chloride profile renders a visual sense of chloride ingress from its source into the concrete, and its relationship with the steel reinforcement. With some approximations, the profile can be characterised by an apparent diffusion coefficient D_a . D_a is then used to project the build-up of chloride ions with time. Various diffusion models are used to predict the service life of concrete structures in marine environments.

The 'charge passed' or 'AASHTO' test is popular because of the very short time required to conduct the test as well as a good repeatability. With a similar test of an accompany sample in curing water, as those suggested by Cao et al. (1996), the modified method is proven to be more chloride-ion specific and hence less binder dependent.

A number of indirect methods such as compressive strength, sorptivity and volume of permeable void have been shown by Sirivivatnanon and Cao (1998) to be valuable techniques but there are also significant limitations in their use.

An apparent chloride diffusion coefficient D_a is a useful parameter to characterise a chloride profile. It is important to recognise that the determination of D_a from Fick's second law was only possible mathematically because many simplifications were adopted. Nevertheless, D_a can be successfully used to study how various binders modify the chloride penetration resistance of concretes. The importance of increasing the length of exposure to the accuracy of estimating long-term D_a was demonstrated. There is, of course, practical limitation of the length of this exposure. Hence, D_a obtained from one-year exposure were used to characterise the resistance of various concretes in this paper.

If the resistance to chloride penetration of various concretes is compared based on equal compressive strength, it is easily seen from Figure 1 that fly ash and silica fume concrete will give the best performance. Concretes manufactured from a general purpose (GP) and high slag (HS) cement resulted in higher D_a . If concretes are compared based on equal water to cement ratio (Figure 2) in the common range of 0.35 to 0.50, there appeared to be very little difference in all four concretes with the exception of the high slag cement concrete. The HS concrete showed noticeably higher D_a than the rest of binders. Finally if comparison is based on concretes of

equal charge passed, as shown in Figure 3, the chloride penetration resistance of each concrete varied significantly. SF and FA concrete are extremely sensitive to the charge passed in the 2000-3000 coulomb and 3500-4500 coulomb ranges respectively. In the more common 1000-2000 coulomb range, both FA and SF concretes gave lower D_s than GP and HS concretes.

6 Conclusion

Chloride-induced corrosion is the prime deterioration factor affecting the durability performance of concrete structures in coastal and marine environments. The resistance of concrete to chloride penetration is well characterised by its chloride profile obtained after a reasonable length of exposure to NaCl solution or synthetic ocean water. Concrete mix design parameters and the type of binder can greatly influence the resistance of concrete to chloride penetration. Based on equal 28-day compressive strength, all structural concrete manufactured from fly ash, slag or silica fume out-performs the GP concrete. Based on equal water to cement ratio in the range of 0.30 to 0.45, the FA, SF and GP concretes are likely to give lower D_s than HS concrete. There is a high level of binder dependency if concrete is judged based on equal charge passed (ASTM C1202).

7 Acknowledgment

Most of the data used in this paper is from a Pacific Power/CSIRO and RTA/CSIRO collaborative research projects. The authors wish to acknowledge the continued support and encouragement from Messrs Peter Heeley, Philip Marsh and Howard Morris of Pacific Power and the Roads and Traffic Authority of New South Wales.

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38 CHARACTERIZATION OF CHLORIDE ENVIRONMENT ALONG A HIGHWAY

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Abstract

A lot of de-icing salt is used on the highway during the winter in Sweden. In order to characterize the salt environment and evaluate the quantity of the chloride ions splashing to the side of the road, the plastic collectors as well as concrete discs were installed at different elevations along the highway Rv40 between Borås and Gothenburg, where a large amount of de-icing salt is spread every year. In this paper the preliminary results measured from the last three winter seasons are presented and the relationships between the measured chloride flow by using the proposed method and the penetration parameters measured by using concrete specimens are discussed.

Keywords: Chlorides, concrete, de-icing salts, highway, road environment.

1 Introduction

In the Scandinavian countries a large amount of de-icing salt is spread on the roads during the winter seasons. According to the information from the Swedish Road Directorate, the annual average dosage of de-icing salt spread on the roads is estimated about 5–10 tons per km road. On some highways, the annual dosage is up to 15 tons per km road! The melt ice-slurry with highly concentrated chloride ions from de-icing salt flows or splashes to the concrete structures by the moving cars and trucks. It is usually believed that the exposure environments for road concrete structures are more severe than those for marine concrete structures submerged in sea water, with regard to chloride initiated corrosion of reinforcement as well as frost scaling damage of concrete. It is even more difficult to model the chloride transport in concrete exposed to the de-icing salt environment by simply using Fick's law. One of

the decisive reasons is the lack of information about the boundary conditions. It is of primary importance to characterize the chloride environment in order to model chloride penetration and further to predict the service life of road concrete structures. In our previous publication (Tang and Utgenannt 1998) a method was proposed to measure the chloride flow along highway. In this paper, the results measured from the last three winter seasons will be presented and the relationships between the measured chloride flow by using the proposed method and the penetration parameters measured by using concrete specimen will be discussed. The work presented in this paper is a part of the Scandinavian project "BTB" - Durability of Concrete Structures Exposed to De-icing Salt.

2 Field exposure station

In the autumn of 1995, a field exposure station near Borås was established through the "BTB" project in cooperation with the Swedish Road Directorate. The field station is situated just on the side of the highway Rv 40, which links Borås with Gothenburg. It is a 150 m long surface covered with gravel only a few meters away from the traffic lane. Along the field station test specimens shaped as short beams are placed in the gravel right behind the safety fence, where they are subjected to the saline road environment, as shown in Fig. 1. In the research project more than 400 specimens from over 120 different concrete qualities are exposed to the severe climate along Rv 40. In connection to the field station there is a local climate station which continuously registers the climatic variations including air temperature, relative humidity, precipitation, wind speed, etc.

At the field station, some plastic chloride collectors as well as concrete discs were vertically installed on the side of the highway with a distance of 3.6 m (Winter 95/96) and 5.8 m (Winters 96-98) from the traffic lane, as shown in Fig. 1. The plastic chloride collectors were used for measuring the chloride flow with the splashed slurry, while the concrete discs were used for measuring the actual chloride penetration into concrete (for Winter 95/96 only), which was previously reported by Wirje and Offrell (1996).

3 Registration of the local climate

The detailed information about the local climate at the field station was elsewhere presented by Utgenannt (1998). The monthly precipitation, temperature and relative humidity are summarized in Figs. 2 to 4 and the monthly spread de-icing salt from 1995 to 1998 are shown in Fig. 5. It can be seen that the total amount of spread de-icing salt per season is about 12-16 tons per km road on this highway, more than the average dosage (5-10 tons per km road).

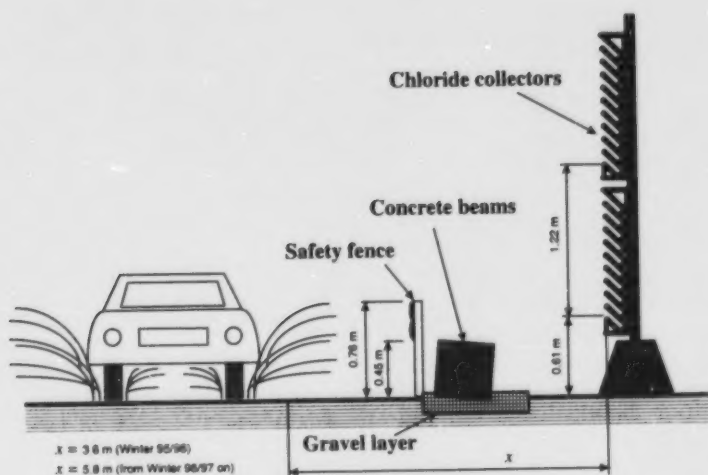
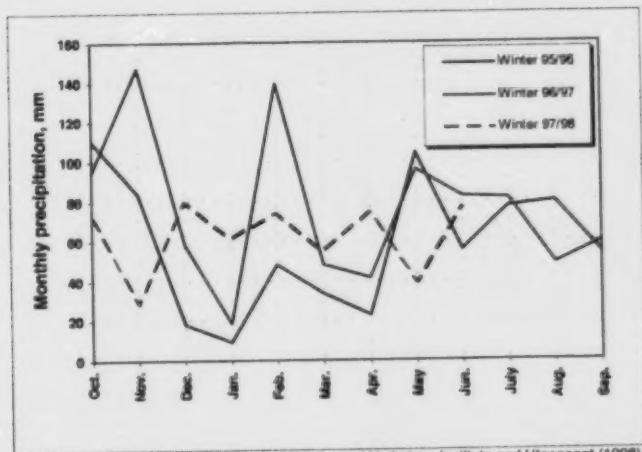
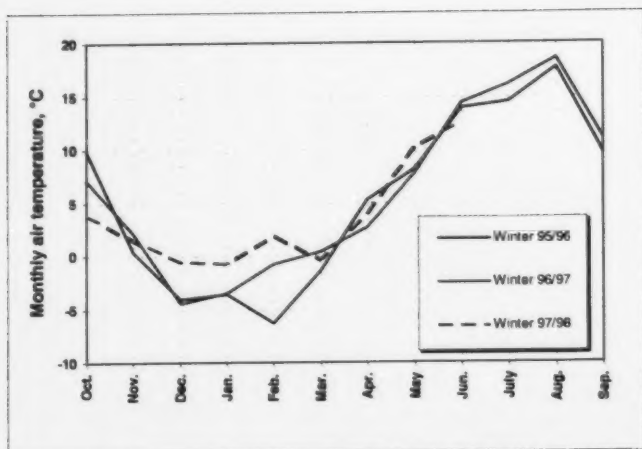


Fig. 1: Schematic of the field station by the highway Rv 40 in Sweden



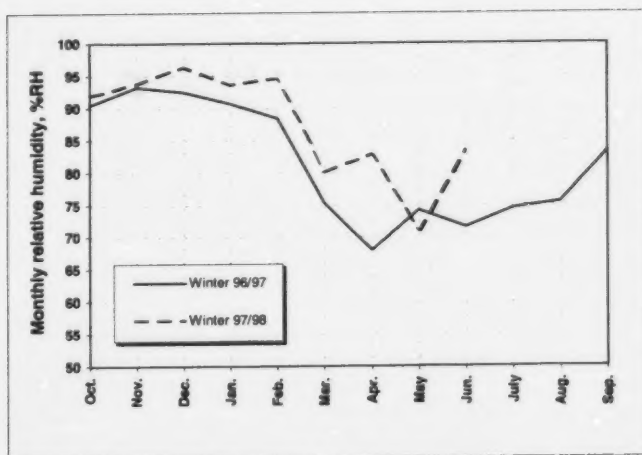
Data source: Swedish Meteorology and Hydrology Institute and Utgenannt (1998).

Fig. 2: Monthly precipitation in the area of Borås



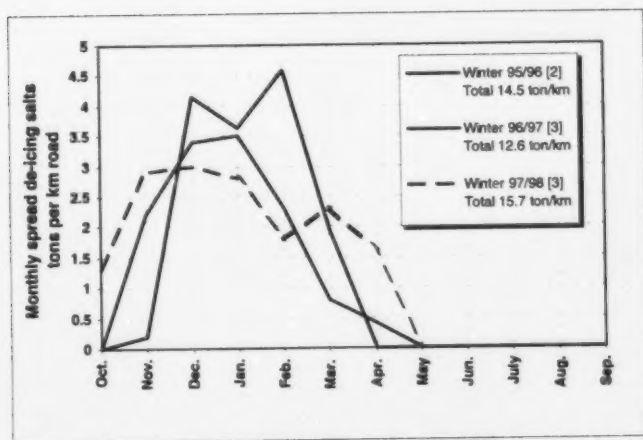
Data source: Swedish Meteorology and Hydrology Institute and Utgenannt (1998).

Fig. 3: Monthly air temperature in the area of Borås



Data source: Utgenannt (1998).

Fig. 4: Monthly relative humidity in the area of Borås



Data source: Wirje and Offrell (1996) and Utgenannt (1998).

Fig. 5: Amount of de-icing salts spread on the highway between Borås and Göteborg

4 Chloride flow into the collectors

The detailed experimental procedure for measuring the chloride flow with the slashed slurry was elsewhere described by Tang and Utgenannt (1998). The principle is to measure the total chlorides accumulated in a collector and then calculate the chloride flow by using the following equation:

$$Q_{Cl} = \frac{W_{Cl}}{A} \quad (1)$$

where Q_{Cl} is the chloride flow into the collectors (g/m^2), W_{Cl} is the collected chlorides (g), and A is the across-sectional area of a collector (m^2).

The measured results of chloride flow are shown in Fig. 6. As expected, the amount of chlorides splashed to the side of highway decreases dramatically with the elevation. The chloride flow is strongly season-dependent. A lowest flow was found in the winter 96/97, especially at the elevations under 1.5 m. It seems difficult to find the relationships between chloride flow and climate parameters.

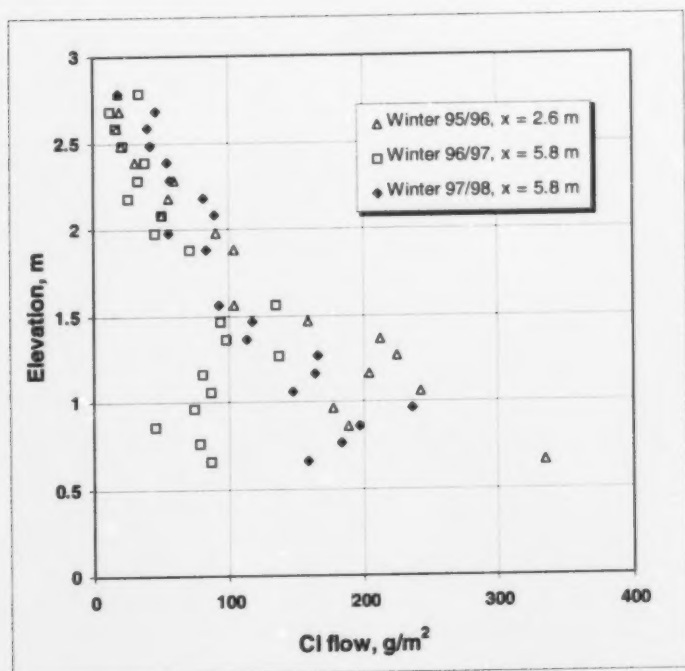


Fig. 6: Relationship between measured chloride flow and elevation

5 Chloride flow into the concrete specimens

In a parallel study, Wirje and Offrell (1996) used another method for evaluating the chloride load along highway. In their method some "standard" concrete discs (w/c 0.45, cement:sand 1:2.4) were mounted parallel to the chloride collectors. After the winter season the chloride penetration profile in each concrete disc was measured. Some chloride profiles measured from the concrete discs are shown in Fig. 7. According to these profiles the chloride flow into the concrete specimens Q_{concrete} could be estimated:

$$Q_{\text{concrete}} = \rho \int C(x) dx \quad \text{g}^2/\text{m}^2 \quad (2)$$

where ρ is the dry density of concrete ($\rho \approx 2200 \times 10^3 \text{ g/m}^3$), $C(x)$ is the chlorides penetration profile (g Cl per gram of concrete), and dx is the depth increment (m).

The calculated results are listed in Table 1, where the surface concentration C_s and the apparent diffusion coefficient D_a , obtained by curve-fitting the profile (after 3 mm depth) to the error-function solution of Fick's 2nd law, are also given.

Table 1: Results of the calculated chloride flow into concrete

Elevation, m	0.44	0.84	1.24	1.64	2.04	2.44	2.84	3.24
Q_{concrete} , g/m ²	38.7	35.7	32.6	25.0	19.7	11.2	8.3	6.2
C_{s1} , % _{concrete}	0.349	0.318	0.307	0.229	0.206	0.164	0.137	0.109
D_{s1} , $\times 10^{-12}$ m ² /s	2.34	2.51	2.41	2.35	2.11	1.27	0.96	0.77

It can be seen that the chloride penetration parameters listed in Table 1 reveal a similar tendency as the chloride flow into the collectors, that is, the magnitude decreases with the elevation.

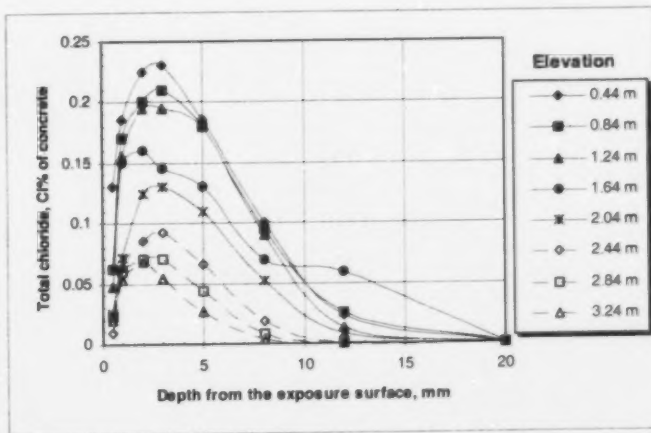


Fig. 7: Chloride penetration profiles measured from the concrete discs after the exposure of Winter 95/96 (Wirje, A and Offrell 1996)

6 Relationships between chloride flow and penetration parameters

6.1 Relationship between Q_{concrete} and Q_{Cl}

Comparing the values of Q_{concrete} in Table 1 with the values of Q_{Cl} (Winter 95/96) in Fig. 6 one can find that the chloride flow into concrete is significantly less than that into the collectors. This should be reasonable, because only partial amount of the splashed chlorides could be able to penetrate into concrete. It was found that the relationship between Q_{concrete} and Q_{Cl} follows a square-root rule in this study, as shown in Fig. 8:

$$Q_{\text{concrete}} = 2\sqrt{Q_{\text{Cl}}} \quad \text{g/m}^2 \quad (3)$$

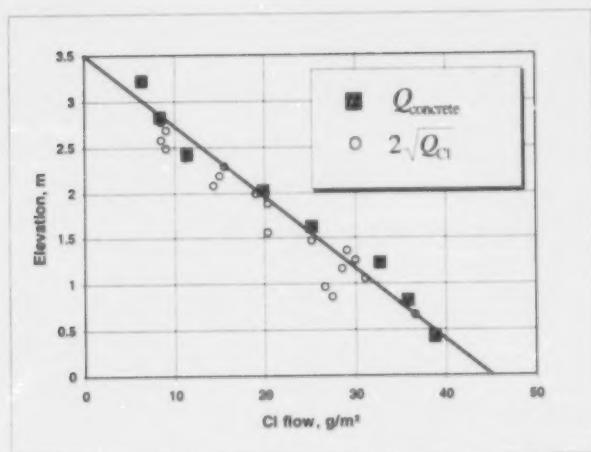


Fig. 8: Relationship between chloride flows measured by the chloride collectors and the concrete discs

6.2 Relationship between C_s and Q_{Cl}

It can be found by regression analysis that the surface chloride concentration C_s is proportional to the logarithm of chloride flow Q_{Cl} , as shown in Fig. 9. This also indicates that only partial amount of the splashed chlorides could stay on the concrete surfaces and further penetrate into the concrete.

6.3 Relationship between D_a and Q_{Cl}

The relationships between the apparent chloride diffusion coefficient D_a and chloride flow Q_{Cl} are shown in Fig. 10. The apparent diffusion coefficient of chloride ions seems proportional to the chloride flow, but becomes constant when the chloride flow is sufficiently large ($> 65 \text{ g/m}^2$ in this study).

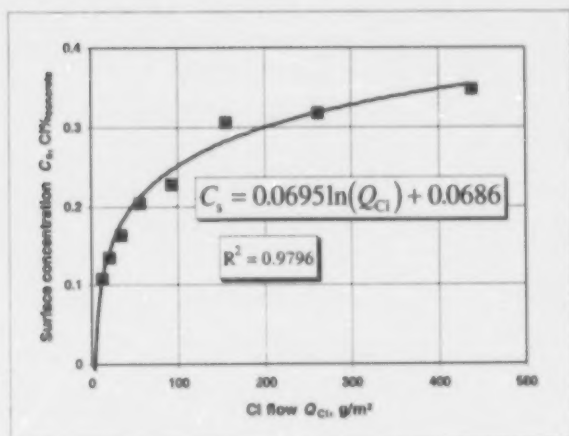


Fig. 9: Relationship between the measured chloride flow and the curve-fitted surface chloride concentration

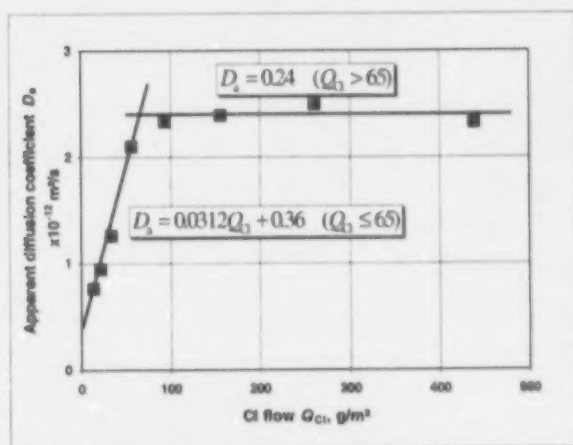


Fig. 10: Relationship between the measured chloride flow and the curve-fitted chloride diffusion coefficient

7 Concluding remarks

As well known, chloride penetration into concrete depends on surface chloride concentration and exposure duration. Since the quantity of chlorides splashed on the concrete surfaces is strongly dependent on many factors, such as the distance between the structure and traffic lane, the elevation, the actual weather, the dosage of de-icing salt, the density of traffic, etc., the boundary conditions for chloride penetration are hardly stable due to different quantities of splashed chloride ions and irregular wet-dry alternations. The curve-fitted parameters C_1 and D_2 as well as the integral parameter Q_{concrete} are a description of the actual chloride penetration profile. Although the relationships between Q_{C1} and C_1 , D_2 and Q_{concrete} are at the present very empirical, the measured chloride flow Q_{C1} indeed tells something about the characters of road environment. It lumps the variations of weather, salt dosage and traffic density and gives an indication of an actual "chloride load" on the concrete structures. Mounting concrete discs along highway is a direct measure for chloride penetration into concrete, but this method is very expensive due to laborious measurements for chloride profiles. While mounting chloride collectors along highway is a simple method for measuring chloride load. The chloride collectors can be reused. Therefore, this is also a relatively inexpensive method. A combination of these two methods could supply informative data for characterizing the environment of concrete structures, and consequently modeling chloride transport in concrete structures, though more data are needed to achieve the latter.

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39 ELECTROLYTIC RESISTIVITY OF COVER CONCRETE: RELEVANCE, MEASUREMENT AND INTERPRETATION

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Abstract

The determination of the electrolytic resistivity of cover concrete is often related to questions of concrete durability. Numerous methods have been developed for a variety of demands, all of which operate on similar basis. Results from three commonly used methods have been compared. Measurements have been performed on concrete specimens with regard to changes in moisture content, the influence of carbonation and the presence of reinforcing bars close to the concrete surface which may lead to misinterpretations of the measured values. It is from these results that the advantages and limits of the three methods can be derived. The conclusions can contribute to a better understanding and interpretation of results obtained from electrolytic resistivity measurements.

Keywords: Carbonation, concrete cover, concrete resistivity, durability, test method

1 Introduction

In order to ensure sufficient durability of concrete structures German standards and guidelines currently in use for the design of reinforced and prestressed concrete structures include only descriptive regulations. Environmental conditions are roughly classified. For each set of these environmental classes the water cement ratio, the cement content as well as the concrete cover is generally specified. Further regulations (e.g. prescribed curing measures) complete the design for durability. This way of designing structures for durability is historically grown and is practised in a similar way all over the world.

Regarding the above mentioned regulations it is not obvious for the designing engineer to comprehend the distinct meanings and valences of the different variables given. Attempts on a European level try to replace the existing design methods by a new

design procedure, based on real material performance. Fundamental aspects of this proof of durability are to consider in detail the environmental conditions, to guarantee a real material performance measured in reproducible compliance tests and to give a comprehensible design for durability based on a probabilistic model and safety concept. Several design models describing the time dependent deterioration processes are reflecting the physical and chemical processes involved, e. g. describing the diffusion of chlorides or the carbonation with regard to the time of exposure. Parameters are reflecting the material properties (concrete, reinforcement, ...), the execution of work and the expected environmental conditions themselves. The incorporation of material compliance tests will ensure that only measured and confirmed material performances are integrated in the durability design.

The resistance of reinforced concrete structures towards rebar corrosion (propagation period) is modelled by a so-called resistivity corrosion rate model (Nilsson and Gehlen 1998). In this model, the electrolytic resistivity of the concrete cover is identified as being one of the most important factors influencing the time dependent progress of rebar corrosion, the corrosion rates being low for high concrete resistivities.

To measure and experimentally specify the electrolytic resistivity of concrete several test methods can be used. Even though all of them are based on a comparable principle, these different test methods can lead to considerable differences in the results. If the electrolytic resistivity shall be used as a design parameter, the exact knowledge of the specific differences arising from the applied test method and the environmental conditions is of great importance. In order to examine and explain the above mentioned difficulties, tests have been performed with different test methods.

2 Investigated test methods

Three relevant test methods for the experimental determination of the electrolytic resistivity ρ of concrete schematically shown in Fig. 1 have been chosen:

- Two-Electrode-Method (TEM)
- Multi-Ring-Method (MRE)
- Wenner-Probe-Method (WER)

The TEM method can be considered being the most elemental measuring method, measuring the alternating current resulting from an implied alternating voltage between two flat steel electrodes. The quotient of voltage and current forms a frequency dependent AC-resistance of the concrete between the two electrodes. The frequency dependent behaviour is usually determined with the so called electrochemical impedance spectroscopy (EIS) for the total frequency spectrum. The complicated measuring equipment makes the EIS test procedure not suitable to use on site. This is the reason why a measurement frequency is fixed, in this case to a value of $f = 108$ Hz, a value confirmed in numerous EIS pre-tests. The TEM with flat steel electrodes as shown in Fig. 1 can easily be applied under laboratory conditions for concrete specimens like cubes or cylinders; however its application on existing concrete structures is limited.

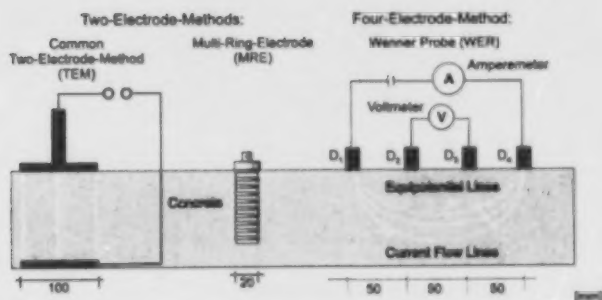


Fig. 1: Test methods used to determine the electrolytic resistivity of concrete

The Multi-Ring-Electrode (MRE) is a special embedded sensor composed of several 2.5 mm thick stainless steel rings which are kept separated at a distance from one another by 2.5 mm thick insulating plastic rings. Cable connections in the sensor enable the resistivity of the concrete to be determined between each pair of neighbouring stainless steel rings by means of AC resistance measurement. The method can thus be considered being a special version of the before described TEM. The measuring process with this sensor offers an advantage over other resistivity measuring processes in that the depth-dependent resistivity can be determined at intervals of 5 mm.

The absolute resistance measured on the test specimen is transformed into a resistivity value using a constant cell factor, as given in equation (1). The cell constant depends on the method and the dimensions of the specimen and can be either determined experimentally (MRE, $k_{\text{cell}} = 0.10$, equation (1)) or by considering the geometry of the specimen (TEM, equation (2)). Due to the fact that the MRE has to be embedded in the concrete, the sensor has to be foreseen before concreting. An implementation of the sensor in existing structures is possible but costly.

The Wenner method works from the concrete surface by using a special arrangement of four equispaced electrodes to imply a constant alternating current between the outer pair of electrodes. The voltage measured between the inner probes leads to the resistance of the concrete. The cell constant for the Wenner-Probe (WER) is found using equation (3), derived from a semi-infinite homogeneous material. Commercially available devices give the resistivity as output straight away with an internal conversion of the determined resistance, independently of whether the requirement of a homogeneous material is given or not.

$$\rho = R \cdot k_{\text{Cell}} \quad (1)$$

$$\rho = R \cdot \frac{A}{l} \quad (2)$$

$$\rho = R \cdot 2 \cdot \pi \cdot a \quad (3)$$

ρ is the electrolytic resistivity [Ωm], R is the electrolytic resistance [Ω], k_{cell} is the cell constant [m], A is the cross section of the specimen [m^2], l is the length of the specimen [m], a is the spacing of the equispaced electrodes [m].

As mentioned above, equation (3) applied for the WER method is only valid if a homogenous semi-infinite material is assumed. Concrete layers of different conductivity limit the use of equation (3) and a theoretical determination of the cell constant turns out to be very difficult. As it is not possible for the user of the WER test devices to influence the cell constant, a misinterpretation is likely. The main parameters of the different devices and test methods used for the test series are listed in Table 1:

Table 1: Test methods for the determination of the electrolytic concrete resistivity

Measuring Device	WER	TEM	MRE
Principle	Four Electrode Method	Two Electrode Method	Two Electrode Method
Frequency	$f = 60 \text{ Hz}$	$f = 108 \text{ Hz}$	$f = 108 \text{ Hz}$
Producer	Proceq (CH)	IBAC-Electronics	IBAC-Electronics
Output	ρ_{WER} in [Ωm]	R_{TEM} in [Ω]	R_{MRE} in [Ω]

In order to determine the resistivity of different concretes, tests have been performed on concrete cylinders under standardized constant environmental conditions (constantly water submerged concrete). As a standard test method, the TEM has been chosen, giving the more realistic, non falsified values for the concrete resistivity. For better comparison, the measured TEM and MRE values (R_{TEM} and R_{MRE}) in [Ω] have been converted into specific values ρ_{TEM} and ρ_{MRE} in [Ωm] with equations (1) and (2).

3 Parameters influencing the electrolytic resistivity of the cover concrete

In the design model (Nilsson and Gehlen 1998), the electrolytic resistivity is taken into account as a measurable material resistance. Its potential value is generally calculated with respect to the used material e. g. the components of the concrete. The type and the amount of binder as well as the water/binder ratio do have decisive influence on the resistivity.

The potential material resistivity $\rho_0(t)$ under constant environmental conditions can be calculated according to equation (4). The first term is a measured potential material resistivity ρ_0 determined at an age of $t_0 = 28 \text{ d}$. The age of the concrete (maturity, degree of hydration) is taken into account by the second factor, where t_{Hydra} is supposed to be the time after which concrete hydration is supposed to be completed when stored under standardized conditions. The third term is a test method factor k_t .

$$\rho_0(t) = \rho_0 \cdot \left(\frac{t_{\text{Hydra}}}{t_0} \right)^n \cdot k_t \quad (4)$$

Further parameters influencing the resistivity $\rho(t)$ are the degree of curing (f_c), the temperature as well as the relative humidity of the environment (f_e), possible carbo-

nation (f_{CO_2}) and possible chloride contamination of the concrete cover (f_{Cl}), leading to equation (5).

$$\rho(t) = \rho_o(t) \cdot f_c \cdot f_e \cdot f_{CO_2} \cdot f_{Cl} \quad (5)$$

The development of $\rho_o(t)$ for twelve different concrete mixes over a period of 91 days when stored under submerged conditions is shown in Fig. 2.

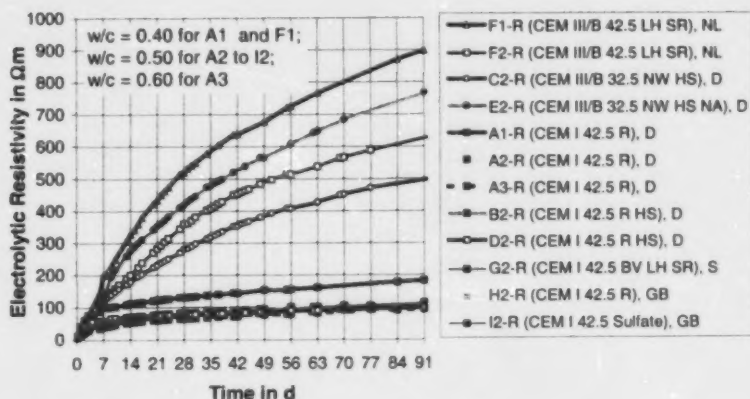


Fig. 2: Concrete resistivity (MRE) vs. time for different concretes

The concrete specimens are cured and stored in such a way, that the above mentioned influencing parameters result in the product of

$$\prod_{i=1}^{n=4} f_i = 1.0 \text{ for standard test conditions (Gehlen and Ludwig 1999)} \quad (6)$$

It can be shown that the electrolytic resistivity is strongly influenced by the choice of binder (curves E2, F2, C2 in contrast to D2, A2, B2, G2, H2, I2) and the water/binder ratio. The age dependency of the measured electrolytic resistivity is obvious.

In order to determine the test method factor, the material resistivity ρ_o determined with the MRE under standardized conditions has been compared to the values measured on the same concretes with the TEM and WER methods. Results are resumed in Table 2.

Table 2: Statistical quantities of the test method factor k_t

Test Method	Mean of k_t	StdDev. of k_t	Distribution Type of k_t
TEM ¹⁾	1.00	-	D
MRE	1.07	0.15	ND
WER	0.68	0.20	ND

¹⁾ MRE and WER values related to the reference method TEM. Reference factor k_{TEM} given as a deterministic value. Standard deviation also observed within a TEM compliance test linked to ρ_a .

4 Influence of moisture content and carbonation of concrete

4.1 Test setup

Besides the above mentioned (constant) differences in output of the three measuring methods even under standardized environmental conditions, differences may arise from concrete properties under practical conditions like moisture content, carbonation, curing, inhomogenities or reinforcing bars. In order to show the importance of these influencing factors, tests have been performed with different three year old reinforced concrete specimens with a carbonated concrete layer for different moisture states of the concrete.

Depth dependent resistivity profiles have been measured with embedded multi-ring-electrodes (MRE), surface measurements have been performed with the Wenner probe (WER) from the concrete surface. Before starting the tests, the specimens (Fig. 3) were stored for one year under dry laboratory climate at 20 °C and 65 % relative humidity. The test was started by a continuous wetting of the concrete surface for a period of 6 days followed by a drying period at similar conditions applied before the wetting period.

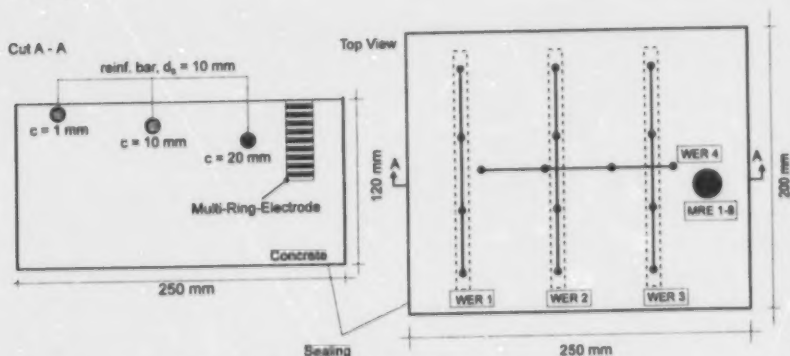


Fig. 3: Specimen design and measurement setup

4.2 Results

4.2.1 Wetting of concrete

The time dependent resistivities measured over 300 hours on the above described specimens is shown in Fig. 4 for the Wenner method (position WER 4) as well as for selected depths with the Multi-Ring-Electrode. The depth of the shown measuring points was chosen as 6, 11, 31 and 41 mm from the concrete surface. The two first measuring points at a depth of 6 and 11 mm are located in the carbonated concrete layer, the 31 and 41 mm values in the non carbonated concrete.

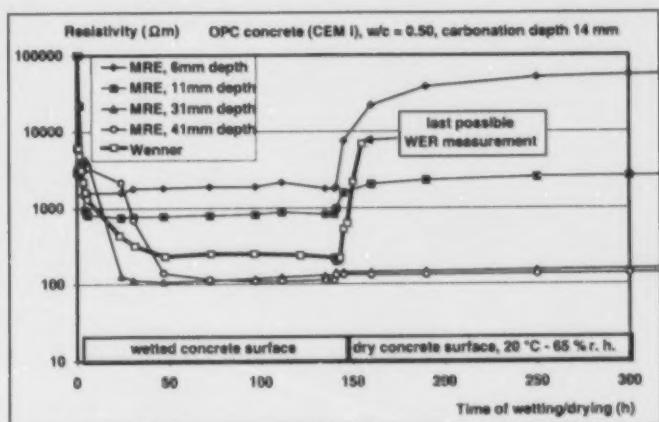


Fig. 4: MRE and Wenner measurements for changing moisture contents of a partially carbonated portland cement concrete

The MRE measurements at different distances from the concrete surface are ideally suited to follow the penetration of the water front at the beginning of the tests. Rather high values are measured immediately at the beginning due to the dry concrete. After wetting of the surface, the values at 6 and 11 mm drop down after a few hours, the 31 mm value reaches its minimum value after 25 hours, at a depth of 41 mm this is only true after a 50 hour wetting time. After 50 hours, constant resistivity values are measured showing much higher values for the carbonated concrete layer of 1000 to 2000 Ωm , the inner non carbonated concrete shows values of only 100 to 150 Ωm . The penetration depth of the water front can be estimated with the MRE measurements: 8-10 mm after 30 minutes, 20 mm after 5 hours and more than 45 mm after 50 hours.

The Wenner measurements (WER) are known to produce an integral value starting from the concrete surface without knowing from the beginning, which concrete layers are included in the measurements. After a 30 minute wetting, the measured Wenner value is 6200 Ωm , after 5 hours 1600 Ωm , a constant value of 230 Ωm is reached after 50 hours of wetting. In order to get a better overview, the Wenner (WER 4) and depth dependent MRE values for these three measurements are resumed in Table 3.

Both measuring methods are clearly influenced by the penetrating water front, whereas the penetration depth cannot be evaluated with the Wenner method. It appears

obvious that the Wenner method at the beginning of the measurements after 30 minutes wetting time is only taking into account a thin wetted concrete layer. As can be seen from the MRE measurements, the resistivity of the deeper concrete layers at this moment is still too high to have a contribution to the measured integral WER value. Due to the internal conversion factor presuming a homogeneous material over the measured area, the WER value in this case is not representative for the cover concrete.

After a wetting time of 5 hours, the penetration depth of the water front has reached 20 mm, this water saturated layer is still too thin and the Wenner values are not representative for the real concrete resistivity. After a period of 50 hours, the penetration depth of the water has reached more than 45 mm. The Wenner resistivity under these conditions can be considered being a good integral value for the cover concrete, even though the higher values of the carbonated OPC concrete layer cannot be detected. Similar effects have been measured on a specimen made with blast furnace slag cement. For this specimen, reliable measurements with the Wenner method can only be performed after a wetting time of 50 hours, good values can be obtained after 24 hours. The results are summarized in Table 4.

Table 3: Comparison of measured MRE and Wenner resistivities (WER 4) for different wetting times for a partly carbonated OPC concrete

Distance from concrete surface (mm)	ρ_{MRE} for duration of wetting			$\rho_{WER 4}$ for duration of wetting		
	30 min (Ωm)	5 h (Ωm)	50 h (Ωm)	30 min (Ωm)	5 h (Ωm)	50 h (Ωm)
6	>100000	1700	1800			
11	>100000	846	780			
16	9700	430	160			
21	6000	4100	150	6200	1600	230
26	4300	4600	125			
31	3500	4000	110			
36	3100	3600	115			
41	2800	3300	140			

Table 4: Comparison of measured MRE and Wenner resistivities for different wetting times for a partly carbonated BFSC concrete

Distance from concrete surface (mm)	ρ_{MRE} for duration of wetting			$\rho_{WER 4}$ for duration of wetting		
	30 min (Ωm)	5 h (Ωm)	50 h (Ωm)	30 min (Ωm)	5 h (Ωm)	50 h (Ωm)
6	>100000	1050	560			
11	>100000	1370	690			
16	>100000	55000	820			
21	>100000	68000	760	6900	2300	730
26	5500	6200	280			
31	3700	4200	280			
36	2300	2600	370			
41	2000	2300	1300			

4.2.2 Drying of concrete

MRE and WER measurements executed on a drying concrete surface revealed that the drying of the concrete is far slower than the corresponding water uptake. An important observation during these test was the fact that after a drying period of only 10 to 20 hours no Wenner measurement could be executed on the specimens, even though the MRE measurements revealed that only the outermost carbonated concrete layer of 5 to 10 mm showed an important resistivity increase. This small layer seems to act as an isolator for the Wenner method making any measurements impossible. This effect has been found to be typical for carbonated concretes.

5 Influence of reinforcement and concrete cover on Wenner measurements

If Wenner measurements shall be performed on reinforced concrete structures, a clear influence of embedded reinforcing steel can be expected due to the known low resistivity of steel. Even though directions for use of the measuring devices indicate to avoid measurements in vicinity of reinforcements, no systematic investigations have been reported on this topic. In order to show the influence of embedded steel bars, additional tests have been realized on the specimens described before. The Wenner probe has been positioned as shown in Fig. 3 in parallel just over steel bars with cover depth of resp. 1 mm (WER 1), 10 mm (WER 2) and 20 mm (WER 3).

Measurements have been executed during the above described wetting tests within the period of $t_1 = 25$ h and $t_2 = 150$ h. As could be expected, a cover of only 1 mm leads to a complete disturbance of the measurements, the whole measurement current can flow through the steel and the measured resistivity rapidly drops to zero. For the cover values of 10 and 20 mm, the measurements are also strongly influenced by the embedded steel bar. Compared to the non influenced WER 4 value, a cover of 10 mm leads to significant reduction of the resistivity. With measurements over a 20 mm covered rebar the reduction is still obvious, but at a lower level. This influence of the concrete cover on the executed measurements after 50 hour wetting is shown in Fig. 5

by calculating the relative resistivity for different concrete cover values, the reference value being the corresponding WER 4 value.

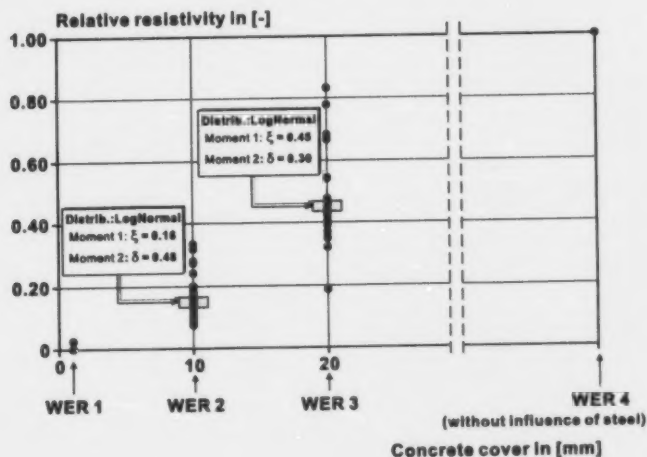


Fig. 5: Influence of longitudinal rebars upon the resistivity value measured with the Wenner probe on wetted concrete

6 Conclusions

When considering the concrete resistivity in order to evaluate durability aspects like chloride diffusion resistance or possible corrosion rates of steel in concrete, measuring methods and test conditions have to be selected carefully. For new concretes, measurements can be foreseen to be executed under standardized laboratory conditions, offering reliable values for all of the three investigated test methods. Difficulties may arise for measurements on existing concrete structures. The Wenner method being one of the simplest method to be applied has been found to be most critical in this context. Representative and reproducible values can only be measured on concretes wetted over a longer period, ensuring that the cover concrete is nearly water saturated. The wetting period strongly depends on the prevailing environmental conditions of the concrete. Tests performed on dry concrete with a carbonated layer revealed minimum wetting times of at least 24 to 48 hours. For normal concretes under outdoor conditions, a prewetting of 24 hours should be on the safe side.

Layer effects caused by carbonated concrete or not sufficiently wetted concrete cannot be detected by the Wenner method, high resistivity values are measured and a misinterpretation is likely. Similar distortions can be caused by reinforcements, measurements should not be performed with the Wenner probe in parallel to reinforcing bars with concrete covers below 30 mm. Under practical conditions, the influence of

reinforcements can be avoided by measuring at several points with changing orientation of the Wenner probe.

Measurements with the Multi-Ring-Electrode can detect any possible layer effect and offer an excellent tool to determine depth dependent concrete resistivity as well as changes caused by moisture or carbonation.

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PART ONE

**DURABILITY OF CONCRETE
MATERIALS AND
STRUCTURES**

**DESIGN AND SPECIFICATION FOR
DURABLE CONCRETE**



40 DESIGN OF CONCRETE TO RESIST CARBONATION

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Abstract

The rate of carbonation in concrete is influenced by both its physical properties and exposure conditions. Therefore, service life prediction of reinforced concrete structures in an environment where carbonation is a probable cause of corrosion of steel in concrete may be made in terms of both the physical properties and the exposure conditions. Investigations in the past have quantified the effect of ambient relative humidity on the rate of carbonation in concrete. Therefore, some attempts have been made to include an environmental stress factor in models to predict the service life of concrete subjected to carbonation. However, when it is the question of service life design of concrete to resist carbonation induced deterioration, a definite procedure is not available at present. This is primarily due to a lack of consensus on parameters to be used for the design. This has prompted the authors to study the relationships which could be used to design concrete for its intended service life when carbonation is a likely cause of deterioration.

Keywords: Air permeability, accelerated carbonation test, carbonation, carbonation rate, compressive strength, durability, service life design.

1 Introduction

Corrosion of reinforcing steel is one of the most critical limit states affecting the durability of reinforced concrete structures. There are a wide range of factors, some internal and some external that create conditions favourable for corrosion to initiate and propagate. One of these causes is carbonation.

Carbonation is the result of carbon dioxide from the atmosphere diffusing into concrete and converting calcium hydroxide and other cement hydrates into insoluble carbonates. This results in a reduction in the pH of the concrete and corrosion of steel in the case of reinforced concrete (Parrott, 1987). Out of 271 cases of deterioration of various types of reinforced concrete structures reviewed in a recent project (British Cement Association, 1997), 46 cases have been due to carbonation induced corrosion of reinforcement. Amongst the 76 cases of buildings reviewed, 33 cases deteriorated due to the effect of carbonation. Therefore, the rate at which carbonation proceeds is important in order to estimate the service life of reinforced concrete structures.

The rate of carbonation can be determined by monitoring the depth of carbonation over a period of time (Kropp and Hilsdorf, 1995; Schiessl, 1988). It generally follows a formula of the form:

$$dc = K\sqrt{t}$$

where dc = depth of carbonation
 K = rate of carbonation, a constant
 t = duration of exposure.

This relationship has been derived from Fick's first law of diffusion and the amount of carbon dioxide required to carbonate the alkaline components in the concrete (Kropp and Hilsdorf, 1995). Although the diffusion coefficient of carbon dioxide through the concrete is assumed to be constant, in order to arrive at the above expression, the constant in the expression encompasses several factors in the carbonation process. These factors are mix proportions, curing regime and age (both providing information on the degree of hydration), material characteristics which control the transport properties, environmental conditions and degree of exposure to carbon dioxide. As a result, the rate of carbonation changes for different concretes and set of conditions to which the concrete is exposed.

The rate of carbonation is obtained from the slope of a plot of carbonation depth against square root of exposure duration (Loo et al, 1994). The main factor that has been found to influence the rate of carbonation is water-cement ratio (Parrott, 1987). As an increase in water-cement ratio increases the porosity and permeability of concrete, this in turn increases the rate of carbonation (Hobbs, 1988). The effect of other mix parameters on the rate of carbonation is not clear in the literature. Therefore, with the present level of knowledge, it is difficult to design concrete for a specific degree of carbonation resistance. In order to deal with this, a detailed experimental programme was carried out by the authors. The results from this investigation are reported and discussed in this paper.

2 Experimental programme

2.1 Test variables

The experimental programme consisted of testing 17 different concrete mixes. Table 1 presents the mix variables and the mass per unit volume of cement, fine aggregate, coarse aggregate and water for the seventeen mixes. The mix

combinations were decided after carrying out trials to check for their viability, i.e. to eliminate honeycombing and segregation. As shown in Table 1, a fine aggregate-coarse aggregate ratio of 0.5 was used in this test series. However, further work is in progress to study the effect of two other levels of fine aggregate-coarse aggregate ratio.

Table 1: Experimental variables and mass per unit volume of constituents

W/C by mass	A/C by mass	Cement content (kg/m ³)	Fine aggregate content (kg/m ³)	Coarse aggregate content (kg/m ³)	Water content (kg/m ³)
0.4	3	563	563	1125	225
0.5	3	533	533	1066	267
0.4	4	468	624	1248	187
0.5	4	448	595	1190	223
0.6	4	429	572	1145	258
0.4	5	400	667	1333	160
0.5	5	387	645	1290	194
0.6	5	374	619	1238	223
0.7	5	360	600	1200	252
0.5	6	340	679	1359	170
0.6	6	328	657	1314	197
0.7	6	319	637	1274	223
0.5	7	302	705	1410	151
0.6	7	295	688	1377	177
0.7	7	287	669	1377	201
0.6	8	266	710	1419	160
0.7	8	260	694	1388	182

Note: Fine aggregate-coarse aggregate ratio = 0.5

2.2 Materials used

Class 42.5N Portland Cement (British Standards Institution, 1991), medium graded natural sand (British Standards Institution, 1992) and crushed basalt 20 mm and 10 mm sizes in the ratio 1:1 were used for all mixes. In order to control the moisture condition of the aggregates, all aggregates were oven-dried at a temperature of 40 °C and allowed to cool for at least 1 day. The saturated surface-dry 30 minute water absorption values (British Standards Institution, 1975) were also measured. These allowed a corrected water quantity to be calculated when batching the materials to ensure that the aggregate in the concrete was in a saturated and surface-dry condition. Tap water at a temperature of approximately 20 °C was used for all mixes.

2.3 Manufacture of test specimens

Four 250x250x100 mm slabs and three 100 mm cubes were cast using each mix. For each mix the required quantities were batched by mass and then mixed together using a pan mixer in accordance with BS 1881: Part 125: 1986 (British Standards Institution, 1986). The test specimens were cast in two layers and compacted using a vibrating table. Full compaction was considered to have been achieved when air bubbles stopped appearing on the concrete surface. After vibrating, the concrete surface was 'rubbed-up' smooth using a metal float and covered with a polythene sheet to prevent evaporation of water from the concrete.

2.4 Curing, conditioning and preparation of test specimens

Approximately 6 hours after the concrete had been cast the moulds were covered with damp hessian. The moulds were stripped next day and the hardened concrete samples were placed in a constant temperature (20 °C) water bath, where it remained for 3 days. These were then wrapped in polythene sheet and transferred into a storage laboratory kept at a constant temperature of 18 °C (± 2 °C) and a relative humidity of 60% (± 5 %). When the specimens were 28 days old, four 50 mm diameter cores and one 100 mm core were cut from each of the four slabs. The cores were then surface dried and the curved surface of each core was coated with Sikaguard, 680 ICOSIT Concrete Cosmetic Acrylic paint. These were then saturated by following an incremental immersion approach and then placed in a drying cabinet at a constant temperature of 40 °C and a relative humidity of 20%. The reason for this was to remove moisture from the surface of the concrete in order to eliminate the effect of moisture on the permeability results and to obtain a mean relative humidity of about 65% in the near surface concrete up to a depth of about 30 mm. The cores remained in the drying cabinet for a period of 14 days and were then transferred to a testing laboratory kept at a constant temperature of 20 °C (± 0.5 °C) and a relative humidity of 50% (± 5 %). After being here for 1 day they were tested.

2.5 Test methods

The three 100 mm cubes were tested at the age of 28 days to determine the concrete compressive strength (British Standards Institution, 1983). The four 100 mm diameter cores were tested for air permeability by using the Autoclam Permeability System (Basheer et. al., 1994) at the end of the conditioning regime. Both the 100 mm and 50 mm diameter cores were then placed in the carbonation chamber.

The carbonation test was carried out at a carbon dioxide concentration of 5% (± 0.2 %) in an environment of 65% (± 1 %) relative humidity and 20 °C (± 1 °C). The depth of carbonation was measured at the start of the test and thereafter each week up to 3 weeks exposure in the chamber. At these times, four cores from each mix were removed from the chamber and the depth of carbonation was determined by splitting the core longitudinally and spraying the split surface with a 1% phenolphthalein indicator solution. After 24 hours the depth of carbonation was measured at three locations at right angles to the exposed surface to an accuracy of 0.5 mm in

accordance with RILEM recommendations in report CPC-18 (RILEM, 1988). An average of the three readings from the four cores was reported as the depth of carbonation after each week's exposure in the chamber. These values were plotted against square root of the duration of exposure and the rate of carbonation was determined for each mix.

At the end of the carbonation test, the 100 mm cores were tested for air permeability and sorptivity, by using the Autoclam Permeability System.

3 Methods of analysis

Two approaches were used to analyse the data. The first attempt was to identify the trends and relationships between the rate of carbonation and both mix parameters and the physical properties. However, no attempt was made to 'quantify' the influence of each of the mix parameters on the measured properties. The second approach was to quantify the effects of the mix parameters, so that the resulting information could be used for the development of the new mix design for carbonation resistance of normal Portland cement concretes. The data were analysed by using the Multiple Linear Regression (MLR) method and nomograms were developed to satisfy this objective.

The main aim of MLR in general was to develop empirical prediction models. Therefore, at the end of this analysis, as well as quantifying the contribution of each of the mix parameters, there were also prediction models for each of the measured properties and ultimately it was possible to use these as the basis for a new mix design procedure. The main steps involved in this overall procedure were as follows:

- (i) Testing normality of experimental data.
- (ii) Developing MLR models.
- (iii) Quantify the contribution of mix parameters on measured properties.
- (iv) Develop prediction models.

A computer package, 'STATVIEW' was used for the analysis of the data. The compressive strength, sorptivity and rate of carbonation results followed a normal distribution reasonably well without applying any transformation to the data, however, a logarithmic transformation was required for the air permeability data.

The MLR models were developed for both 'mass ratio' (i.e. water-cement ratio and aggregate-cement ratio) and 'mass/unit volume' (i.e. cement content, fine aggregate content, coarse aggregate content and water content, all in kg/m^3). In addition, a predictive model for rate of carbonation based on compressive strength, air permeability and sorptivity was developed. Before developing the MLR models, the correlation between rate of carbonation and each of the variables used in the analysis was determined. This is reported in Table 2. Then, starting with the variable which had the highest correlation, the analysis for the development of the MLR model was carried out. Table 3 presents the improvement of the model with the addition of each variable in the MLR models. The MLR analysis resulted in the following empirical models (note that additive models were developed in all cases):

$$\begin{aligned} \text{RoC} &= \beta_0 + \beta_1(\text{W/C}) + \beta_2(\text{A/C}) + \varepsilon_i && \text{(mass ratio)} \\ \text{RoC} &= \alpha_0 + \alpha_1(\text{CC}) + \alpha_2(\text{FAC}) + \alpha_3(\text{CAC}) + \alpha_4(\text{WC}) + \varepsilon_i && \text{(mass/unit volume)} \\ \text{RoC} &= \gamma_0 + \gamma_1(\text{Fcu}) + \gamma_2(\text{API}) + \gamma_3(\text{SI}) + \varepsilon_i && \text{(properties)} \end{aligned}$$

where

RoC	=	Rate of carbonation
β_0, α_0 and γ_0	=	intercept
β_1 and β_2 or α_1, α_2 and α_3 or γ_1, γ_2 and γ_3	=	multipliers for the independent or explanatory variables
ϵ_i	=	random error associated with the variate

Table 4 presents the intercepts and multipliers for the MLR models for rate of carbonation.

Table 2: Correlation between rate of carbonation and variables used in the analysis

Description	Variable	Coeff. of determination, R^2
Mass ratio	W/C	0.829
	A/C	0.192
Mass/unit volume	Cement content, CC	0.275
	Water content, WC	0.092
	Fine agg. content, FAC	0.01
	Coarse agg. content, CAC	0.014
Properties of concrete	Compressive strength, Fcu	0.792
	Air permeability, API	0.429
	Sorptivity, SI	0.642

Table 3: Development of MLR models for rate of carbonation

Type of MLR model	Variables included	Explained (%)	Improvement (%)	Unexplained (%)	Coeff. of multiple determination R ²
Mass ratio	W/C	82.92		17.08	0.838
	A/C	83.80	+0.88	19.20	
Mass/unit volume	CC	27.48		72.52	0.825
	CC+WC	79.69	+52.21	20.31	
	CC+WC+CAC	79.70	+0.01	20.30	
	CC+WC+CAC+FAC	82.51	+2.81	17.49	
Physical properties	Fcu	79.19		20.81	0.801
	Fcu+SI	79.93	+0.74	20.07	
	Fcu+SI+API	80.10	+0.17	19.90	

Table 4: Intercepts and multipliers for the MLR models for rate of carbonation

Type of MLR model	Explanatory variables	Intercept	Multiplier	Coeff. of multiple determination, R^2
Mass ratio	W/C	-7.567	26.225	0.838
	A/C		-0.206	
Mass/unit volume	CC	-164.74	0.0553	0.825
	CC+WC		0.3274	
	CC+WC+CAC		0.3535	
	CC+WC+CAC+FAC		-0.033	
Physical properties	Fcu	12.18	-0.157	0.801
	Fcu+SI		0.735	
	Fcu+SI+API		1.280	

4 Discussion of results

4.1 Influence of mix parameters on rate of carbonation

Figure 1 shows the rate of carbonation for different water-cement ratios and aggregate-cement ratios for a constant fine aggregate-coarse aggregate ratio of 0.5. It can be seen that the effect of water-cement ratio is to increase the depth of carbonation for all the aggregate-cement ratios. Generally this effect increased towards higher aggregate-cement ratios. Apart from experimental variation, the rate of carbonation remained almost constant at different aggregate-cement ratios for all water-cement ratios. The increase in the rate of carbonation due to the increase in water-cement ratio can be related to increased porosity and carbon dioxide transport. This result was reported in several publications (Parrott, 1987; Hobbs, 1988). The effect of aggregate-cement ratio observed here appears to be in contradiction to results reported by other researchers (Papadakis et al., 1991), that there was an increase in depth of carbonation due to an increase in aggregate-cement ratio. However, the data obtained from the literature was for concretes for which the water-cement ratio increased when the aggregate-cement ratio increased. As the trend reported in figure 1, was repeated at four different water-cement ratios, it is justifiable to conclude that there is no significant effect due to an increase in aggregate-cement ratio on the rate of carbonation.

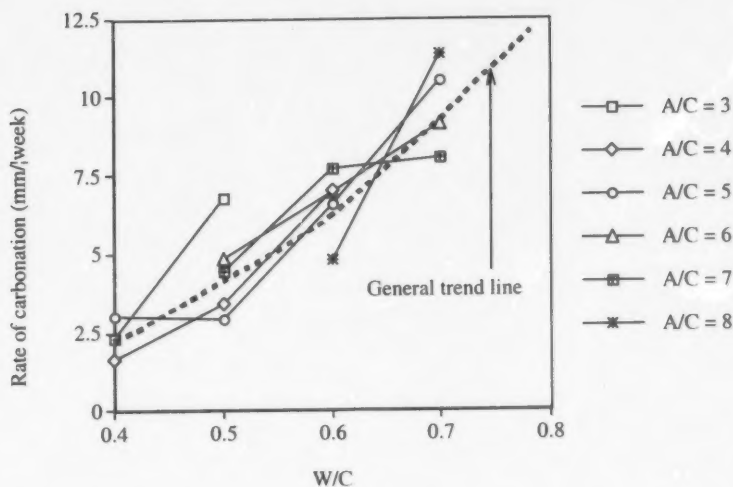


Fig. 1: The effect of water-cement ratio and aggregate-cement ratio on rate of carbonation

The dotted line in Figure 1 shows the overall effect of water-cement ratio on the rate of carbonation. At each of the water-cement ratios, the effect of aggregate-cement ratio is not very significant. The spread is mainly due to the water-cement ratio and this alone explained about 83% variation in rate of carbonation (Table 3). There was only a marginal improvement in the MLR models by the addition of aggregate-cement ratio (0.88%).

The dependence of rate of carbonation on mass/unit volume can be seen in Table 3. Both cement and water content together (i.e. W/C effect) contributed to about 80% variation of the rate of carbonation. Other factors, such as the amount of fine aggregate and coarse aggregate in the mix, have no significant effect on the rate of carbonation. When the results of mixes with fine aggregate-coarse aggregate ratio of 0.3 and 0.7 are available, this analysis will be repeated and the true effect of both these variables quantified.

Figure 2 reports the effect of water cement ratio and cement content on rate of carbonation. For each aggregate-cement ratio, the cement content varied slightly when the water-cement ratio was changed. However, the main effect that can be seen in this Figure is that the main variation in rate of carbonation is due to the change in water-cement ratio. For the whole range of cement content tested, the rate of carbonation decreased marginally with an increase in cement content. However, this cannot be considered as overly significant. Another trend which can be obtained is that, for a given water-cement ratio, the increase in aggregate-cement ratio resulted in a significant decrease in cement content and this, in turn, decreased the rate of

carbonation (shown as dotted lines in Figure 2). However, there was kink in the trend for all water-cement ratios. The information available at present cannot explain the reason for this sudden jump in the trend. The reduction in rate of carbonation is considered to be due to the reduction in calcium oxide with the decrease in cement content (Papadakis, 1991).

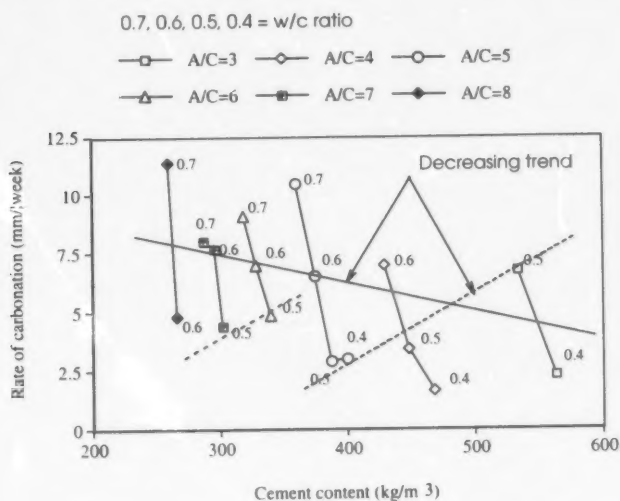


Fig. 2: Effect of w/c ratio and cement content on rate of carbonation

4.2 Dependence of rate of carbonation on measured properties of concrete

The depth of carbonation is correlated with compressive strength, air permeability and sorptivity in Figures 3, 4 and 5 respectively. In Figure 3, there is a very good correlation between the rate of carbonation and the cube compressive strength, as indicated by a value of 0.823 for the coefficient of determination. This accounted for about 79% of the variability in the MLR model (Table 3). (Note that an exponential relationship is presented in Figure 3, however, the regression coefficient in Table 3 is for the linear regression analysis.)

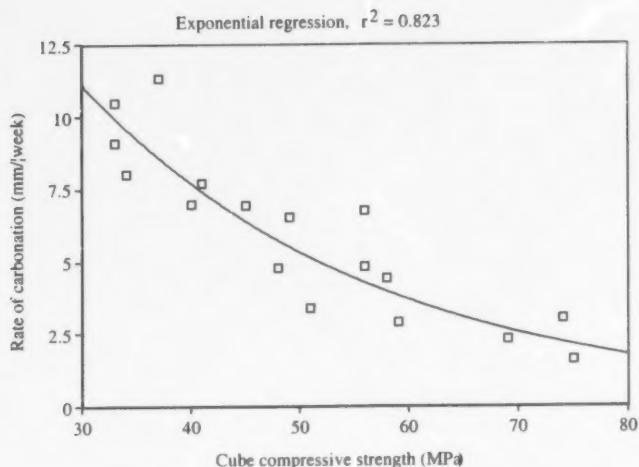


Fig. 3: Dependence of rate of carbonation on cube compressive strength

The correlation between the rate of carbonation and both the air permeability and the sorptivity in Figures 4 and 5 is not as good as that obtained for compressive strength. The effect of both conditioning and mix parameters on permeation properties differs from that on cube compressive strength. As the effect of these are not studied in this investigation, only general trends are given emphasis at present. The improvement of the MLR model in Table 3 with the addition of both the sorptivity and air permeability is insignificant when comparing this with the effect of compressive strength alone.

4.3 Development of design curves

The above discussion would suggest that in order to design normal Portland cement concrete mixes for carbonation resistance, a simplified approach is to control the water-cement ratio. However, it should be noted that this needs to be verified for various aggregate grading and types of aggregate. It is known that the water-cement ratio controls the compressive strength of normal Portland cement concretes. As a consequence, if concrete is designed for a specified compressive strength, an estimate of resistance to carbonation may be made.

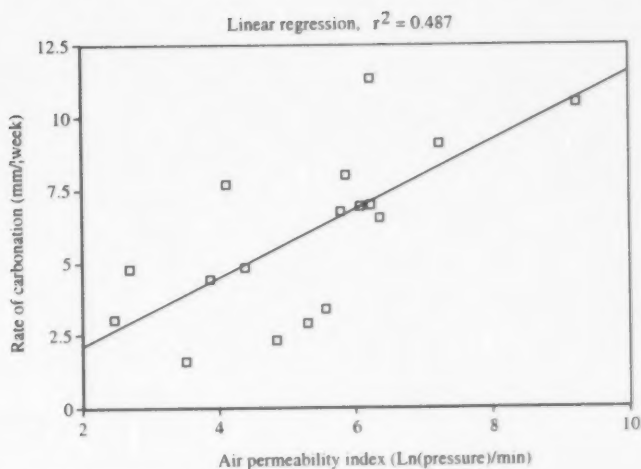


Fig. 4: Dependence of rate of carbonation on air permeability

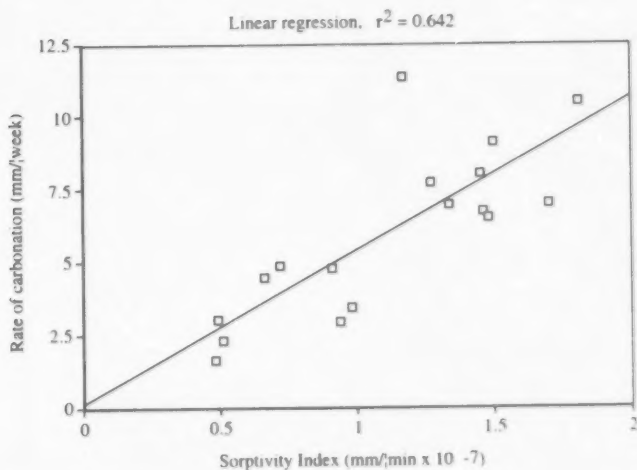


Fig. 5: Dependence of rate of carbonation on sorptivity index

If service life design of concrete to resist carbonation is required, the MLR models in Table 4 could be used. Figure 6 illustrates the prediction model for rate of carbonation on the basis of water-cement ratio and aggregate-cement ratio.

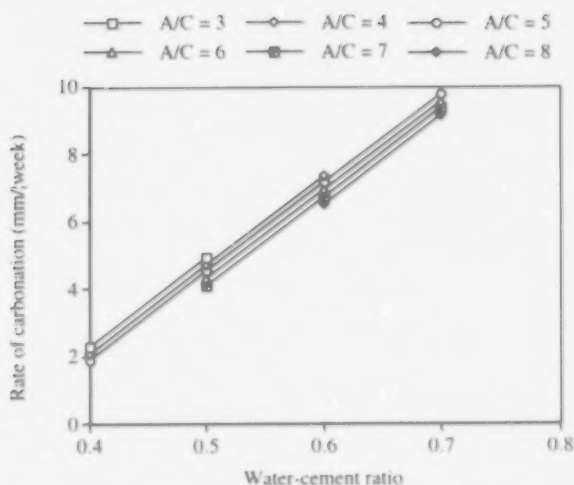


Fig. 6 Design graph for carbonation resistance of normal Portland cement concrete

Figure 6 would suggest that, if the findings based on accelerated carbonation testing can be validated for normal exposure conditions, the rate of carbonation can be used to determine the mix proportions which would ensure a specified service life of reinforced concrete. This is because, for a given depth of cover, the rate of carbonation could provide the time before which carbonation reaches the reinforcement. Further research in this area is in progress at present at Queen's and it is expected that a suitable procedure for the service life design of concrete for carbonation resistance can be developed.

5 Conclusions

On the basis of the results presented in this paper, the following conclusions have been reached.

- i) The rate of carbonation of normal Portland cement concrete is primarily influenced by the water-cement ratio and other factors have only a marginal effect. Therefore, concrete can be designed to have a specific carbonation resistance based entirely on the water-cement ratio. However, this should be verified for different types of and gradings of aggregate.
- ii) The rate of carbonation from accelerated carbonation testing can be used for the service life design of concrete in an environment where carbonation is a probable cause of deterioration. However, there is a need to relate the results

of the accelerated test to those of natural exposure trials before this approach can be used.

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41 QUALITY CONTROL PROBLEMS IN HIGH PERFORMANCE CONCRETE STRUCTURES

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Abstract

In the past decade, the use of high performance concrete for the construction of structures exposed to severe freeze/thaw and corrosion environments has become commonplace. The use of silica fume, high-range water reducing agents, corrosion inhibitors and other materials have become quite commonplace, as durability of concrete components becomes more and more important in the proportioning of concrete mixtures for structural design. Unfortunately, this familiarity of the industry to these materials has led to problems in their execution. Through the use of case studies, this paper explores the difficulties that can arise during the execution of high performance concrete designs. Case studies are presented which deal with problems encountered during or after construction, which affect the long-term performance of building systems. In the major case study presented, the life cycle implications of deviation from the design are discussed in terms of reduced resistance to abrasion and chloride ion ingress. Quantitative as well as qualitative analyses are both employed to describe the loss in structure lifetimes. This paper addresses contractor experience, pre-qualification, pre-construction testing and ongoing quality assurance and the significance of each of these attributes in the construction of durable buildings.

Keywords: concrete, quality control, diffusivity, specifications, silica fume, freezing and thawing, scaling

1 Introduction

In recent times, we have asked more of the concretes that are used in building construction. Initially, this started with concretes of high strength and rapidly developed to concretes that had other properties which were desirable, and which were well outside of the range normally encountered. Extensive research on the use of mineral and chemical admixtures, cement chemistry and construction practices has resulted in an arsenal of high performance concretes (HPC) to address the needs of designers, owners, maintainers and operators of concrete structures in achieving performance of concrete structures for various applications.

Unfortunately, the construction process does not end with the specification and mixture proportioning. During construction, deviations from the specifications or failure to pay attention to routine quality control and/or the specific requirements for these high performance concretes can affect the long-term performance of the structure. The lack of sufficient quality assurance testing can also lead to problems in the execution of the construction.

The purpose of this paper is to use case studies to illustrate how problems can arise in high performance concrete construction projects. Where applicable, and where mathematical models exist, the impact of these observed conditions on the life cycle of the structure is reported. Where mathematical models do not exist, a qualitative estimate of the decrease of lifetime is given. Not every case is a fault due to construction practices. Errors in testing are discussed as well, in that they are treated with some degree of severity at the time the tests are performed. While not necessarily having an impact on the life cycle of the structure, they do point out one of the problems of quality control of high performance concretes, specifically that methods used for testing ordinary concrete may not be adequate to test modern HPC.

2 Case studies

2.1 Case 1. Parking structure in North America

In early 1996, an addition was made to an existing five-story parking structure. This addition consisted of placement of three additional floors. The parking structure is a cast-in-place concrete frame, with post-tensioned slabs.

The concrete specification required the concrete to be: purposefully air-entrained; to replace cement with silica fume at a replacement level of 7 percent; to contain a corrosion inhibitor; to contain a water-reducing agent to have; a water/cement ratio of 0.35; and to contain a high range water-reducing agent to get the material in place. No submittal requirements based on any measure of performance of this concrete were requested, which the designers intended to resist the penetration of chloride ion.

During construction the lower five floors of the structure remained in service. Shortly before the structure was completed, it was noted that a distinctive "wave" pattern was evident in most of the concrete slabs. These waves appeared to coincide with the location of reinforcing steel within the concrete. Further examination indicated that at numerous locations where the bars terminated in the slab, the bars

moved to such an extent that they were exposed at their ends. A coring and testing program was undertaken to determine the cause(s) of the unusual surface condition that was observed and a review of the construction documents was made to supplement this study.

Fieldwork indicated that the crests of the "waves" were located between the reinforcing steel bars and that the troughs were located immediately above the reinforcing steel. Anecdotal evidence indicated that during operation of the garage, there was significant vibration of the addition, which was easily detected by standing still on the slab.

The air-void content and characteristics were determined both for cores extracted from the structure as well as cylinders obtained for the purpose of routine strength quality control. Additionally, rapid chloride permeability was determined on cores overlying and away from the reinforcing steel. The results of typical tests are presented in Table 1. The RCP values were, on average, about 88 percent higher for the concrete above the steel rebars.

Table 1: Rapid chloride permeability above and away from reinforcement

Test Area Number	Rapid Chloride Permeability (Coulombs)	
	Above Steel	Away From Steel
1	1985	940
2	1508	647
3	1381	1039
4	1166	750
Mean	1510	844

The air void analysis showed that while the total air content did not change significantly between the concrete over the top of, and between, the reinforcing steel bars, there was a marked change in the air void characteristics. Typical pairs of results representing concrete separated by a few inches, above and between the reinforcing steel, are presented in Table 2 along with data obtained from a cylinder taken at the time of placement for routine quality control testing.

Although both air void systems are consistent with our understanding of what is required for frost resistant concrete, it is apparent that the significant change in total air void content, bubble size and spacing was due to the additional vibration which was caused by the movement of steel bars while the concrete was still plastic, resulting in a coarsening of the air void structure and reduction in the air content in the concrete around the bars. This was supported by petrographic examination, which indicated the presence of bleed water pockets below the reinforcing steel and aggregate around the steel.

The final piece of the "puzzle" was the examination of cylinders that had not been placed into the structure, but rather, retrieved from the site for routine quality control testing. These cylinders had not undergone the additional vibration given by the steel within the structure, and were found to contain an air void content in excess of 14 percent.

Table 2: Air void characteristics of concrete above and away from reinforcement

Test Area Number	Air Void Property ASTM C457	Above Steel	Away From Steel
1	Air Content, %	9.9	11.2
	Specific Surface, mm	14.96	14.57
	Spacing Factor, mm	0.19	0.17
2	Air Content, %	8.7	10.8
	Specific Surface, mm	20.67	24.61
	Spacing Factor, mm	0.15	0.13
3	Air Content, %	8.4	12
	Specific Surface, mm	21.26	23.62
	Spacing Factor, mm	0.13	0.09
4	Air Content, %	8.3	9.4
	Specific Surface, mm	15.16	17.13
	Spacing Factor, mm	0.19	0.17
Test Cylinder	Air Content, %		14.2
	Specific Surface, mm		17.72
	Spacing Factor, mm		0.15

Additionally, the air void system was noted to be extremely dispersed, with a high specific surface and a small spacing factor. Based on the petrographic examination, many of the voids were seen to be sufficiently small that surface tension would have prevented them from being detected using the conventional pressure meter specified in the documents and commonly used to test air void content of plastic concrete (Hover 1988).

Although rapid chloride permeability provides a qualitative description of the ability of chloride to penetrate into the concrete, a relationship can be developed between diffusivity and the resistivity of concretes (Northwood and MacDonald 1994). Using this correlation, the mean diffusivity for concretes above and some distance away from the reinforcing steel was calculated and is presented in Table 3.

Table 3: Estimated diffusivity based upon average rapid chloride permeability values (Table 1)

Location	RCP Value Coulombs	Estimated Diffusivity $\times 10^{-13} \text{ m}^2 / \text{sec}$
Above Steel	1510	90
Away From Steel	844	8.7

This diffusivity can be used to predict the penetration of chloride ion into the concrete (Hooton and McGrath 1995). If we assume that the clear concrete cover to the reinforcement is 50 mm, a surface chloride ion content of 400 ppm and that the chloride ion content required to initiate corrosion of reinforcing steel is 0.025 percent

by weight of concrete. The predicted chloride ion profiles and the time to reach the threshold level can be calculated. The calculated chloride ion content at the 50 mm depth (the specified cover depth) is presented in Figure 1 as a function of exposure time (years) and location.

For the concrete above the steel rebars, the chloride ion threshold level for initiation of corrosion would be reached in approximately 20 years, whereas similar levels of chloride ion would not be reached for 100 to 200 years in the concrete away from the steel rebar. In addition, the high measured air void content of the concrete surface has led to concerns over the abrasion resistance of the upper surface of the concrete.

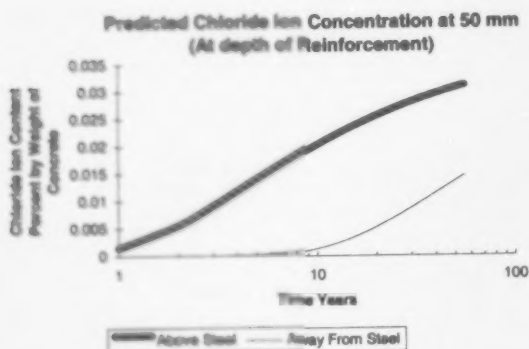


Fig. 1: Predicted chloride ion profile based on Fickian diffusion

2.1.1 Summary

In summary, the interaction and use of numerous admixtures created a very high air content concrete with very fine bubbles that could not be detected with a pressure type air meter. The construction details and operation of the parking structure during construction resulted in additional consolidation over the reinforcing steel that reduced the ability of the concrete to resist penetration of chlorides. This significantly reduced the expected lifetime of the structure.

2.2 Case II. Parking structure in North America

This case study discusses both the impact of the failure to perform quality assurance testing to measure compliance with the performance based specifications, and reliance on indirect measurements of these important properties.

A parking structure was constructed in the late 1980's in North America using concrete containing 8 percent silica fume. The project specifications required that the mix proportion submittal include Rapid Chloride Permeability (RCP) as measured by AASHTO T 277, indicating the result should be below 1000 coulombs passed. The ready mix producer used a laboratory pan mixer to design the concrete mixture and submitted results of RCP testing performed by an independent testing lab as required

by the specification.

The mixture proportions were approved for construction. Quality control testing at the job site consisted of slump and air contents of the plastic concrete and compressive strength of the cylinders cast on site at 7, 28 and 56 days. This testing was done and no problems were encountered.

Approximately 3 to 4 years after the structure had been put into service a number of cracks were noted in the suspended slabs. Further investigation resulted in the discovery of corrosion of the embedded reinforcing steel at cover depths of up to 25 mm. A petrographic study of the concrete was undertaken which involved optical microscopy as well as testing cores obtained from the project. This study revealed that the concrete had a RCP value over 1500 coulombs. This did not meet the project submittal requirements. The optical petrography clearly indicated the underlying reasons for the deviation between the submittal and the concrete delivered to the site. The concrete used in construction of the garage contained silica fume at the appropriate proportion, however, that silica fume was found not to be dispersed throughout the matrix. As a result, although there was an increase in strength, the microstructure had not been modified such that the concrete was as resistant to chlorides as it would have been had the silica fume been dispersed, i.e. $RCP < 1,000$ coulombs.

As a preventative measure, a coating was placed on the silica fume structure that had not originally been intended to be used to bolster the system's resistance to the penetration of chloride ions.

2.3 Case III. Poor testing practice

This case study discusses the impact of the method of curing of samples on the results obtained, and indicates that testing practices themselves can be the source of perceived problems.

A parking structure was under construction in the upper Midwest in 1997. The project requirements of the concrete were that it contain 8 percent silica fume, have a compressive strength of 68.9 MPa (10,000 psi), and be air-entrained. Construction commenced early in the spring and proceeded throughout the hot summer months. A compressive strength problem was noted with the concrete commencing approximately mid-June. The 56-day strength of 10,000 psi was not being met, and there were concerns with the 7- and 28-day results of concretes, which had been cast later. A review was made of the test data, indicating that the majority of the concrete placed after approximately the first week of May was showing problems with respect to strength. After holding a meeting on site, involving not only the construction team, but the design team and the financial backers of the project as well, it was determined that a coring program should be undertaken to determine the actual strength of the concrete in the structure. At the same time, an engineering study was undertaken of the concrete samples.

The results of the coring indicated that the in-situ concrete had an average strength over 80 MPa and that no cores were taken which fell below the minimum specified 56-day strength. The engineering study indicated that there was extensive microcracking occurring within the cylinders, which was not visible within the cores obtained from the structure. A review of site conditions indicated that while the concrete cylinders had been stored out of direct sunlight, there had been no control over their temperature. Testing practices were changed to include curing the silica

fume modified cylinders directly in water tanks. Immediately, an increase in strength of compressive strength cylinders was noted, and the alarm that had been generated by the testing laboratory reports began to dissipate.

Clearly, when handling high performance concrete, problems can arise from all aspects of ordinary practice, and are not limited solely to those aspects that are associated with the real problems.

2.4 Case IV. Problems with air entrained concrete

This case study discusses the use of admixtures and their interactions. Repairs were undertaken to a large structure exposed to freezing and thawing in the saturated condition. The project specifications required a thin concrete overlay on existing pavement slabs. A number of admixtures were used, including an air-entraining agent, a water reducing agent, a super plasticizer, and others. The concrete was required to be delivered throughout the summer months in the hot weather at a temperature of no greater than 60 degrees Fahrenheit. As a result, some of the mixing water was added as ice and liquid nitrogen was used to cool the concrete mixture itself.

During trial batching, it was noted that two of the admixtures were interfering with each other. Specifically, the air void content of the concrete was difficult to maintain. In the end, a triple dose of air-entraining agent was required to meet the total air content specifications.

During construction, it was noted that air void contents at the job site were becoming very high. In response, the ready mix supplier reduced the air content by cutting back on the admixture dosage rate of air-entraining agent. This process continued until the ready mix supplier was not adding any air-entraining agent, and the concrete at site was still maintaining total air void content between 4 to 7 percent.

When the structure was put into service over the winter, the surfaces scaled completely. Engineering evaluation indicated that the total air content of the concrete had been 4 to 7 percent, as measured by pressure meters. The air void system was extremely coarse, with an L-bar in excess of 600 micrometers. The existing overlays had to be removed, and replaced with conforming materials.

In this case, the lack of clear air void structure characteristics in the specification and the lack of understanding on part of the ready mix supplier, lead to the observed failure.

2.5 Case V. Scaling resistance

In the spring of 1997, a large number of concrete flatwork durability problems were noted to be occurring in the upper Midwest. The extent and frequency of this damage was on an unprecedented scale and was being blamed on the use of "foreign" cements, on the use of fly ash, and on a winter which was said to be the "worst ever" for concrete.

In response to the numerous complaints, the Aggregate and Ready Mix Association of Minnesota sponsored a study that was performed by Dr. Mark Snyder of the University of Minnesota (Snyder 1998). This study was supplemented by work done by one of the authors (K.A. MacDonald) and by Mr. Richard Stehley of American Engineering and Testing. A number of core samples were solicited from the ready mix producers in Minnesota and were to be selected to represent typical conditions. A petrographic analysis of these cores was performed and the underlying

reason or reasons for the damage were determined.

It could be suggested that concrete flatwork exposed in Minnesota is high performance concrete, because it is required to resist extreme freeze/thaw conditions. The results of the Aggregate and Ready Mix Association of Minnesota study were illuminating. Over 70 percent of the concretes examined had problems with the air void system. Forty-eight percent of these had been overworked, and showed a low air void content and poor bubble size distribution at the surface only, however, 21 percent had low or no air entrainment. Additionally, 40 percent of the samples had a water/cement ratio in excess of 0.45. The complete tabulation of the controllable factors affecting performance in the ARM study is presented in Table 4.

The ARM study concluded that there were numerous factors that led to the observed problems. While the severe winter conditions, combined with a very dry summer, had singled out marginal concretes, the main conclusions were that; the concretes had been constructed using aggregate which included non-durable particles, which unfortunately are unavoidable in some cases in Minnesota; had used inappropriate mix designs with water to cementitious ratio higher than 0.45 and low cementitious contents and in some cases, had poor air void structures; and had been subject to poor finishing and to inadequate curing. Recommendations from this report were that water/cement ratios be lowered, air contents be increased, curing practices and finishing practices be improved and understanding of the role of aggregate in surface distress should be developed.

**Table 4: Controllable factors affecting performance in ARM study
(Snyder 1998)**

Contributing Factor	Percent of Samples
Air Entrainment Problems	70
Low air at surface only	48
Low air throughout sample	15
Non air entrained	6
Finishing Problems	61
Inadequate curing	61
Low Cement Content	55
Long Transit Time	42
High Water/Cement Ratio	39
Non Durable Aggregate	30
Deicing Chemicals	6
Poor Jointing	3
ASR	3

3 Conclusions

Through the use of case studies, we have identified problems that occur during construction that can have an impact on the lifetime of the structure. These problems are independent of the designs, which in every case were adequate for the environmental exposure. It is important to remember that the execution of high performance concrete structures does not end with preparation of specifications, but is necessary to be followed through the construction process to result in durable structures that will perform as expected.

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42 CONCRETE, THE DURABLE BUILDING MATERIAL

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Abstract

When the decision was made to construct the huge gravity base structure for the Hibernia oil extraction project off the coast of Newfoundland, the construction material that was chosen was concrete. Further, the material of choice for the Confederation Bridge was concrete. In another totally different vein, the paving material for the recently constructed Highway 407 around Toronto was also selected to be concrete. These three projects, widely different in character and function, all had the same underlying specification...a need to satisfy long term performance criteria. The watchword is DURABILITY.

Concrete is recognized as a material which can be made to last, whether it has to face pack ice pressures, pounding waves, freezing and thawing conditions or heavy vehicular traffic. However, there have been known cases where concrete systems have deteriorated prematurely. Parking structures represent a well known example, where inappropriate material specifications, design criteria and construction practices often combined to compromise long term resistance to freeze/thaw exposure and de-icing salt corrosion. Stucco cladding systems represent another area where less than ideal material specifications, detailing and construction practices have combined to cause unacceptable performance consequences. Recognizing serviceability criteria, any concrete mix can be proportioned to provide the necessary properties in both the fresh and hardened states to perform satisfactorily. Durability is the watchword. Long term performance of any concrete related construction can be successfully achieved through careful attention to material specifications, application and details.

Keywords: concrete, durability, portland cement, stucco, weather resistance.

1 Introduction

It is the intention of this paper to show that concrete is one of the most durable building materials available to mankind. This ability to resist deterioration from the harshness of the surrounding environment is a property of not only cast in place or precast concrete but of all concrete products that are made with portland cement. Such products include concrete masonry units (concrete blocks, bricks and paving stones), mortars, grouts (means to achieve soil and waste stabilization), roller compacted paving and building cladding options, such as rendering and stucco.

It cannot be denied that many structures have been built, especially within our lifetime, which have not performed in a manner consistent with the premise of long life. This paper looks at the more common failures with regard to durability, examines what went wrong and makes suggestions to avoid their recurrence in the future. Included in the review of indicated improvements to ensure the integrity of concrete systems with regard to durability is a look at recent developments in the field of high performance concrete along with practical applications.

2 Examples of durable concrete structures

Although portland cement as we know it dates back only to 1824 when Joseph Aspdin obtained a patent for his product, there are examples in the world of concretes made with other forms of hydraulic cements which were built long before this date and which still survive today. One of the best examples is the domed roof of the Pantheon in Rome, which was constructed over 1800 years ago. Its arched span on concrete ribs measures 43.3 metres, not an insignificant structure for any age. The structure is still standing today. The concrete contained a cement made from a mixture of ground lime and volcanic ash found near Pozzuoli, Italy, from which the name pozzolanic cement is derived.

There are innumerable other examples in the world today of concrete structures that have successfully withstood the ravages of time.

In 1908, Thomas Edison took out a patent for a system of cast-iron moulds for a monolithic concrete house. Edison's system made it possible to form walls, floors, stairways, roof, bath and laundry tubs and conduits for electric and water services in one single casting operation. His house is still in existence.

In Winnipeg, the construction of concrete paving for streets also began in 1908. While many of these have been resurfaced and now function as rigid bases, there is one example of one of these early streets in a residential neighbourhood is still exposed and performs satisfactorily. There are many other concrete streets in Winnipeg that were built in the early and mid fifties that are still performing well without resurfacing.

These are only a few examples that illustrate the point that concrete can be made to be durable. There is no reason today that all concrete should not be made to last.

3 Present day durable structures

Turning to the present day, we recognize many significant structures for which durability, as a vital property, was incorporated in the design.

The Confederation Bridge, linking Prince Edward Island with New Brunswick is an excellent example. As a design-build-operate-transfer contract, the onus was on the contractor to develop the most economical structure possible, not only in consideration of the initial capital cost, but also from the point of view of long term maintenance. Attention was given to known factors affecting corrosion of reinforced and post-tensioned concrete:

- chlorides
- concrete alkalinity
- oxygen availability
- concrete resistivity

In consideration of these factors, it was concluded that the most effective way to protect the structure against corrosion would be to use a High Performance Concrete in combination with adequate cover.

Other considerations, although nevertheless leading to an assurance of durability, led to the choice of High Performance Concrete for the Hibernia Gravity Base Structure. The caisson's exterior walls form an ice shield 1.4 metres thick that is designed to absorb iceberg impact. The internal walls form a system of enclosed cells of which some were designed for the storage of oil. It is vital that the base slab and interior walls of the structure remain completely watertight, so crack control was a major factor in the design.

Durability, in terms of wear resistance in addition to freeze/thaw resistance, was a major factor in the selection of concrete for the paving material for the recently built Highway 407 across the city of Toronto.

4 Recognized inadequacies

There has been a justifiably large amount of concern in recent years about crumbling infrastructure. No construction material has been immune to this unsightly, potentially dangerous and financially burdensome phenomenon, and concrete is recognized to have performed poorly in many cited instances. It may seem paradoxical to claim that concrete can be one of the world's most durable construction materials when we know that the world is replete with examples of dismally distressing failures.

Parking garages probably represent the prime example of unsatisfactory performance of concrete construction. There is no need to discuss the immense costs that have been associated with parking garage repairs over the past several years. Our concern here is why this problem had to happen in the first place. Because of the immensity of the problem, enough investigative work has been documented over the

years so that improvements in design and construction techniques could be prescribed in the relevant CSA Standards. The single cause of the problem is the liberal use of salt on roads and city streets. Migration of chloride ions into reinforced concrete does not create a healthy environment for material stability. CSA Standard S413 addresses this reality in parking structure design by offering a choice of protective treatments, including increased cover to the top reinforcement, to ensure long term serviceability of the structural system.

Highway bridge structures have also been recognized to have suffered undue early deterioration, chiefly as a direct result of chloride induced corrosion. The liberal use of de-icing salts has generated a monumental repair bill. But here again, an understanding of the chemical processes leading to corrosion has brought about improvements not only in design and construction, but in the materials from which the structural concrete is made.

In both bridge construction and parking garage construction, High Performance Concrete is becoming increasingly accepted. Its improved water-tightness forms an effective barrier to the ingress of salt laden water. In many cases, the increased cost is offset by associated savings, either through the reduction of structural material quantities or through the use of less stringent protective measures.

In the above discussion, the concern has been focussed mainly on de-icing salts and the inevitable chloride ion. There are other corrosion inducing concerns, however, that have come to the forefront in recent years and which also have been addressed. Chiefly among these is the phenomenon of alkali-aggregate reactivity and the associated delayed ettringite formation. Further discussion on this subject appears in the next section of this paper.

5 Durability requirements

Durability refers to the ability of concrete to resist deterioration from the environment or service in which it is placed. Properly designed concrete should endure without significant distress throughout its service life.

In order to ensure that concrete will be durable, attention has to be paid to a number of important issues. Design Codes and Standards include provisions specifying limitations regarding water/cement ratio and percentage of air-entrainment for various degrees of exposure. Other provisions include requirements for chloride content, cover to reinforcement, curing period and protective measures during construction. These documents, however, do not provide guidance on minimum cement or cementitious materials contents for various anticipated uses. It is a well known fact that richer concrete mixes are more durable than leaner mixes.

Recommendations for durability and scale resistance include:

- A slump of 80mm or less
- A cementing materials content of 335 kg/m³ or more
- Proper finishing after bleed water has evaporated
- Adequate drainage with a slope of 1% or more

- Minimum 7 days moist curing at 10° C
- A minimum 30 days drying before exposure to freezing.

The addition of blast-furnace slag, silica fume, or fly-ash may have a negative effect on the de-icer scaling resistance of concrete when a major part of the portland cement is replaced by these supplementary cementing materials. Percentage limits are specified in CSA Standard A23.1-94 in an attempt to prevent deterioration in service. When combinations of portland cement and supplementary cementing materials are used, the concrete should be tested to confirm adequate durability.

5.1 Air entrainment

The percentage and distribution of entrained air in the mix is an important factor in assuring durability against freeze/thaw deterioration. Tests are conducted in the field on the plastic concrete to verify the percentage of entrainment and laboratory procedures are carried out on hardened concrete to assure that the air-void system is appropriate for the particular environment.

5.2 Chloride content

Concern with chloride-induced corrosion of reinforcing steel has led to the need to monitor and limit the chloride content of reinforced concrete. Limits are established on the water-soluble chloride-ion content of hardened reinforced concrete. It is important to note that the test method determines the total water-soluble ion content of the concrete, including those locked into the aggregates. In cases where chloride in the aggregates is suspected, separate testing of the aggregates would be required and a correction made to the total content to determine the actual amount of free chloride ions.

5.3 Alkali-aggregate reactivity

Alkali-aggregate reactivity is a type of concrete deterioration due to a reaction between the active mineral constituents of some aggregates and the sodium and potassium alkaline components in the concrete mix, which are usually present in the portland cement.

All natural rocks react with the cement paste to a certain extent which is generally beneficial as this contributes to good bond between the paste and the aggregate. However, in some instances the reaction can result in excessive expansion and cracking of concrete that is exposed to moisture. This cracking leads to a reduction in strength while at the same time allowing an increase in potential saturation with its inherent reduced freeze/thaw durability.

A thorough discussion on the causes and effects of alkali-aggregate reactivity is beyond the scope of this paper. Nevertheless, it must be appreciated that the risk of sudden structural failure in concrete due to alkali-aggregate reactivity is almost non-existent. In fact, much of the concrete in Canada that was made with reactive aggregate remains in service. However, the resulting formation of cracks may interfere with serviceability through ingress of moisture and/or salts leading to other forms of deterioration. There is an excellent set of guidelines on the subject in Appendix B of CSA Standard A23.1, Concrete Materials and Methods of Concrete

Construction. This provides information on testing of aggregates for reactivity, a listing of known potentially problematic aggregate sources in Canada and measures to mitigate alkali-aggregate reactivity. These measures take the form of chemical and mortar bar tests on suspect aggregate supplies and the reduction, if necessary, of the total alkali content of the concrete mix by the partial replacement of the portland cement by suitable supplementary cementing materials. Today, selective quarrying meets the principles laid down in these guidelines.

6 Stucco

No discussion on the durability of cement based construction materials would be complete without a serious look at stucco cladding systems.

There is a very serious misunderstanding that has developed in recent years concerning the performance of stucco cladding systems in multi-family residential construction, particularly in West Coast locations. In fact, the serviceability of portland cement stucco has been proof-tested through long use in extreme environments. But, as with any other construction material, failure to follow specifications or to use acceptable construction practices will inevitably compromise the weather resisting capabilities of any building system.

In addition to the stucco material and the way in which it is applied, building details have a definite influence over subsequent performance. Of particular concern in this regard is the modern tendency to minimize roof overhangs which are vitally effective in shedding roof accumulated runoff away from exterior wall surfaces.

6.1 Admixtures

Admixtures in stucco mixes should not be used indiscriminately because they generally are not needed and are often detrimental to subsequent performance. Of special concern are forms of plasticizers that are not recognized by industry established specifications. Stucco mixes are very sensitive to admixtures. The absorptive property of the substrate also has a marked influence over the effectiveness of an admixture.

Commonly acceptable admixtures include:

- air-entraining admixtures
- accelerators
- bonding agents
- colouring pigments

Unacceptable admixtures include:

- antifreeze compounds
- water repellents
- detergents

6.2 Application

Care and attention has to be exercised in preparing the substrate for stucco. In sheathed frame construction, waterproof building paper or felt should always be placed over the sheathing followed by the metal reinforcement, which should be attached to the supports rather than to the sheathing. Attention has to be paid to accommodating building movements through correctly located control joints to avoid inducing internal stresses that can lead to cracking. This includes paying attention to details at windows and door openings as this is crucial for successful performance.

If prepared and applied properly, portland cement stucco should form an excellent weather resistant exterior wall covering that should perform satisfactorily in a wet climate.

7 High performance concrete

This is a term usually used to describe concretes made with carefully selected high quality ingredients, optimized mixture designs, and which are batched, mixed, placed, compacted and cured to the highest industry standards. Typically such concretes will have low water/cementing materials ratios which impart a degree of superiority to the following desirable in-place properties:

- High strength
- Abrasion resistance
- Low permeability
- Low absorption
- Low diffusion coefficient
- High resistivity
- Resistance to chemical attack
- High modulus of elasticity
- High resistance to freeze/thaw damage
- Volume stability
- Inhibition of bacterial growth

From this list it can be seen that the emphasis for many applications may not necessarily lie in compressive strength, but rather on other properties having more relevance to the particular application.

Earlier in this paper we were introduced to a few outstanding examples of successful applications of high performance concrete. This material is clearly a great improvement over previous formulations of high quality concrete, made possible by the use of modern admixtures and supplementary cementing materials. It is a pity that the construction industry in general is inhibited by the traditional first cost approach to assessment of viability rather than a more rational long term approach based on a reliable life-cycle cost analysis. Ultimately, the superior qualities of high performance concrete, especially in regard to durability, will lead to its wider acceptance.

8 Conclusion

Durability refers to the ability of a concrete based material to resist deterioration from the environment or from the service in which it is placed. Any portland cement based construction material can be proportioned to provide necessary properties in both the fresh and hardened states to perform satisfactorily for its expected service life.

This paper has presented examples of concrete applications that have successfully withstood the ravages of time. It has also looked at recent concrete construction projects for which durability was a major consideration in the design and measures were taken to specifically ensure that these particular structures would experience a long, trouble free life.

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43 DURABLE CONCRETE THROUGH SKIN TREATMENT WITH CPF

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Abstract

A good protective outer layer for concrete structures may be obtained with CPF- Controlled Permeability Formwork. A study of various CPF systems was carried out to ascertain the benefits of CPF compared to concrete cast with traditional Portuguese formwork. Various tests were carried out, such as surface hardness, absorption by capillarity, water penetration, accelerated carbonation, accelerated chloride ion diffusion, abrasion and mercury porosimetry tests with assessment of porosity values and pore size distribution curves. All results lead to the conclusion of the beneficial use of CPF systems to obtain an optimum skin cover over the reinforcement.

Keywords: Concrete, durability, Controlled Permeability Formwork-CPF, tests.

1 Introduction

Durability of concrete structures depends on a well compacted strong concrete skin with low permeability, low diffusivity, no cracking, an adequate thickness over the reinforcement and, of course, on the environmental actions on the structures. A good protective outer layer of concrete structures may be obtained in many ways such as simplicity in design, adequate mix with low water/binder ratio, adequate binder, good compaction and sufficient curing. Controlled Permeability formwork (CPF) is also a solution to obtain a good protective outer layer of the concrete, Fig.1.

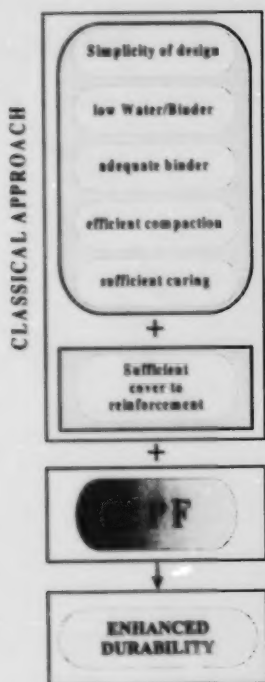


Fig 1: Enhanced durability using CPF

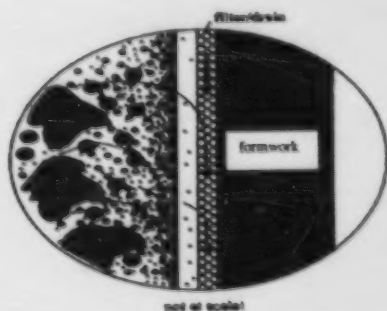


Fig 2: How CPF works during compaction

CPF consists of using a textile liner on the formwork, allowing air bubbles and surplus water to drain out, but retaining cement particles. This enables the water-cement ratio of the outer layer to become very low and the concrete to hydrate to a very dense surface skin as the filter makes enough water available at the right time to activate optimum hydration.

A study of various CPF systems was carried out to ascertain the benefits of CPF compared to concrete cast with traditional portuguese formwork. Traditional portuguese formwork is made of pine wood boards attached to each other. The influence of the use of various systems of CPF on two brands of concrete was also evaluated.

The research programme involved various tests namely surface hardness, absorption by capillarity, water penetration, mercury porosity tests with assessment of porosity values and pore size distribution, accelerated carbonation, chloride ion diffusion and abrasion tests.

Two concrete mixes were used for this research programme, as shown in Table 1.

Table 1: Concrete composition

Kg/m ³	Mix 1	Mix 2
Cement Type II 32,5	250	420
Coarse aggregate 15/25	587	796
Fine aggregate 5/15	391	89
Coarse sand	587	796
Fine sand	391	89
Water	183	184
W/C	0.73	0.44

Several concrete walls of $100 \times 100 \times 20 \text{ cm}^3$ were cast and each divided into vertical strips corresponding to different types of formwork. The types of formwork considered were traditional formwork, with no treatment, which was considered the control formwork type and two CPF formwork systems:

- CPF:T, a system in which the filter/drain (Terram) was tensioned over traditional formwork and
- CPF:Z, in which the filter/drain, Zemdrain MD1 (DuPont), that needs no tensioning was simply attached to traditional formwork.

2 Tests

2.1 Surface hardness

Schmidt-hammer tests were carried out 28 days after casting and results obtained for the strips of concrete cast with CPF systems were 53 to 102 % higher compared to the same concrete cast with traditional formwork (pr EN/ISO 8045 was followed, to carry out these tests).

When these tests were completed caps were drawn out from each wall and used for the remaining tests.

2.2 Absorption tests

Absorption tests consisted of dipping pre-dried cores a few millimetres into water (75mm diameter) and weighing them at intervals until maximum absorption.

The cores were prepared so that water would only be absorbed through the surface corresponding to the formwork as a 10mm ring of epoxy resin had been applied to the cylindrical surface next to the formwork face, Fig.3.

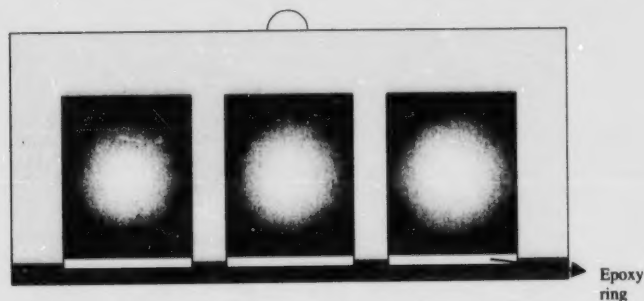


Fig. 3: Absorption by capillarity tests

The water absorption, in grams per square unit of formwork face area, was evaluated at time intervals starting from the moment of dipping the cores. This enabled plotting an absorption curve for each core (g/cm^2 versus $\sqrt{t(hours)}$). Various absorption parameters were calculated and compared such as Initial Absorption (1 hour), which was estimated to be 59 % to 83 % lower in CPF concrete compared to the same concrete cast traditionally and the Absorption Coefficient S , calculated at 4 hours of testing which was 62 % to 89 % lower in CPF concrete compared to traditionally cast concrete.

2.3 Water penetration tests

For Water Penetration tests UNE-83-309-90 (24 hours of pre-drying) was followed in general. For 150 mm diameter $\times$ 200 mm length cores, a reduction of 49 % to 95 % of the average penetration depth was observed, comparing the same concrete obtained with CPF systems and traditional formwork. For 75 mm diameter $\times$ 100mm length cores, the range of reduction was 73 % to 81 %.

2.4 Accelerated carbonation tests

Accelerated Carbonation tests were carried out using chambers with 5 % carbon dioxide and Relative Humidity (RH) of approximately 65 %, where 75 mm diameter cores were kept for at least 3 months. These cores were previously coated with epoxy resin on all surfaces, except the ones corresponding to formwork. Some results are shown in Fig.4. Reduction of the depth of carbonation was observed to be 66 % to 99 % lower in CPF concrete compared to the same concrete cast traditionally.

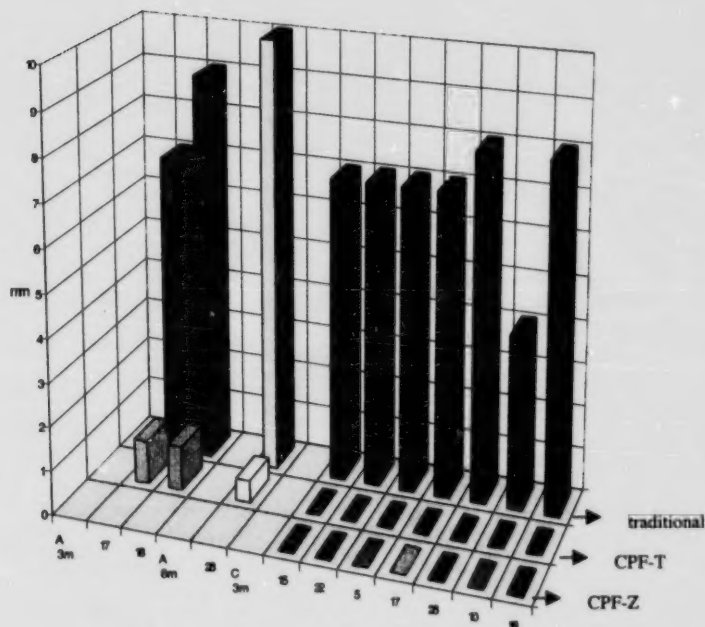


Fig. 4: Accelerated carbonation depths for cores from various test-walls

2.5 Abrasion tests

Abrasion tests were conducted on an Amsler-Laffon testing machine and abrasion was 3 % to 14 % lower in CPF concrete compared to the same concrete cast traditionally. These tests were technically very difficult to carry out when using formwork made of pine wood boards, the surface is rough and it has to be levelled out before.

2.6 Chloride ion penetration

For Chloride Ion penetration tests cores coated with epoxy resin on all surfaces except the ones corresponding to formwork, were alternately immersed (3 or 4 days) in a highly concentrated NaCl solution (15%) and dried in a ventilated oven at 40° C, 3 or 4 days, during a total of at least 53 days. Then, for each core, concrete dust was drilled out from at least four holes at step-depths of 1cm and the chloride ion concentration was analysed chemically. This enabled determining the chloride ion concentration cross section for each core. Examples are shown in Fig.5.

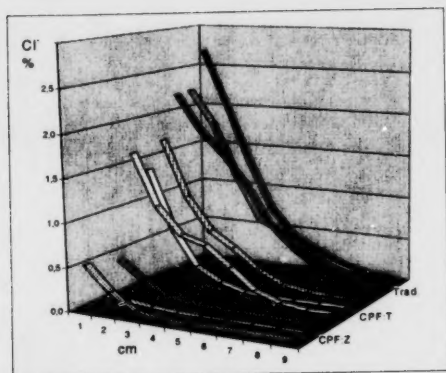


Fig. 5: Chloride Ion penetration cross-sections for three cores cast traditionally (further back), three cast with CPF:T (middle) and three cast with CPF:Z (front), of brand 1 concrete

Applying the Diffusion theory it is possible to estimate D , the diffusion coefficient and C_s , the surface chloride ion concentration using a solution for Fick's 2nd Law. D was observed to be 38 % to 92 % lower in CPF concrete compared to the same concrete cast with traditional formwork. C_s was 34 % to 73 % lower on CPF concrete compared to traditional.

2.7 Mercury porosimetry tests

Mercury porosimetry tests consist of using a special equipment called a Porosimeter, where a sample, in this case of concrete, is introduced. The sample is surrounded by mercury and submitted to growing pressure (p) which can be related to the corresponding minimum pore diameter (d) of the pore structure which is mercury penetrated. Porosimetry tests enable the assessment of porosity values, and also provide pore size distribution curves. These tests were carried out in two phases, the first corresponding to tests carried out on the top 4 mm layer and the second phase on the underlayers at 4 mm step depths up to a maximum, depth of 20 mm.

Cores, 20 mm diameter, were drilled out from the concrete walls at the same level in each vertical strip corresponding to each formwork type. These cores were then sawn into approximately 4 mm slices. Porosity values of the top 4 mm layer, according to formwork type, are shown in Table 2.

Table 2: Average porosity values in the top 4 mm layer

Concrete brand	Wall	Number of samples	Average porosity value		
			traditional	CPF-T	CPF-Z
1	B	2+2+2	13.0%	7.6%	8.4%
2	A	5+6	11.6%	5.7%	no tests
	C	10+8+10	11.3%	7.7%	7.6%

The porosity values in the top 4 mm layer are 32% to 51% lower in CPF concrete compared to the same concrete cast traditionally. Some pore size distribution curves for brand 2, top 4 mm concrete layer, are shown in Fig. 6 for wall A, and Fig. 7 for wall C.

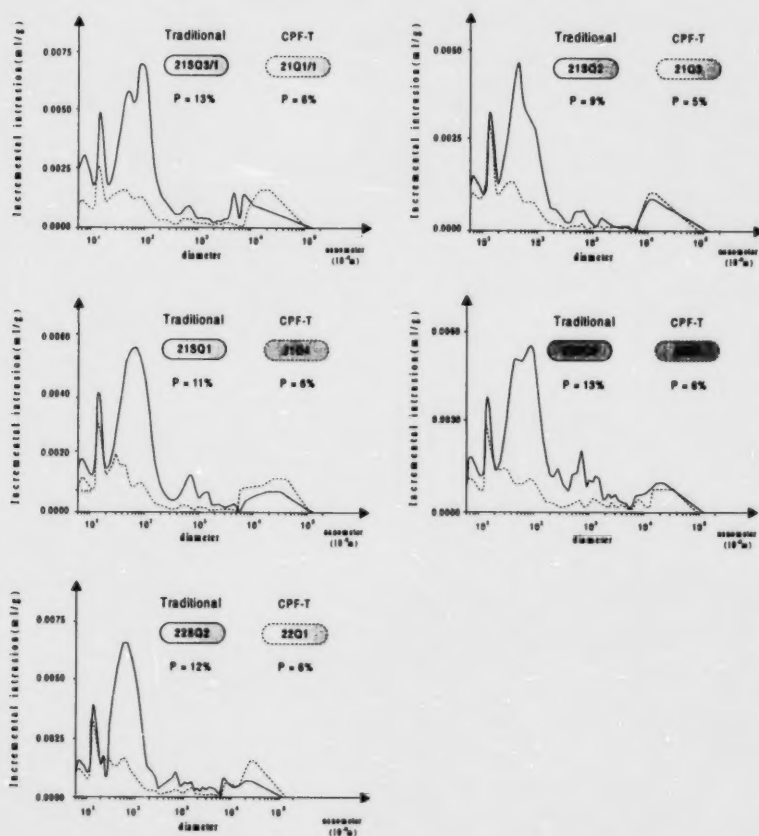


Fig. 6: Some pore size distribution curves for brand 2 top 4 mm concrete layer for wall A

The distribution curves can be analysed with special focus on the capillary pores with diameters approximately between 60 nm ($1 \text{ nm} = 1 \times 10^{-9}$) to just over 10^4 nm (Setzer, 1995). These are the relevant pores in terms of durability because macropores, which result from mixing, compaction and aggregate size distribution, usually are not connected to the pore system and micropores are too small concerning fluid transport.

Analysing Fig. 6 and comparing CPF:T curves with traditional cast concrete curves (trad.), there is a significant reduction in the quantity of capillary pores. In fact the modal value, around 90 nm for all trad. curves, reduces to about 15 nm when considering corresponding CPF:T curves. Therefore, it seems obvious that when CPF:T is used, there is a clear reduction of relevant pores in terms of durability as the pore distribution curves appear to "move" in the direction of smaller pores. This phenomena is similar to that observed by Prof. Hilsdorf when comparing concrete with reducing water/binder ratio (Hilsdorf, 1989).

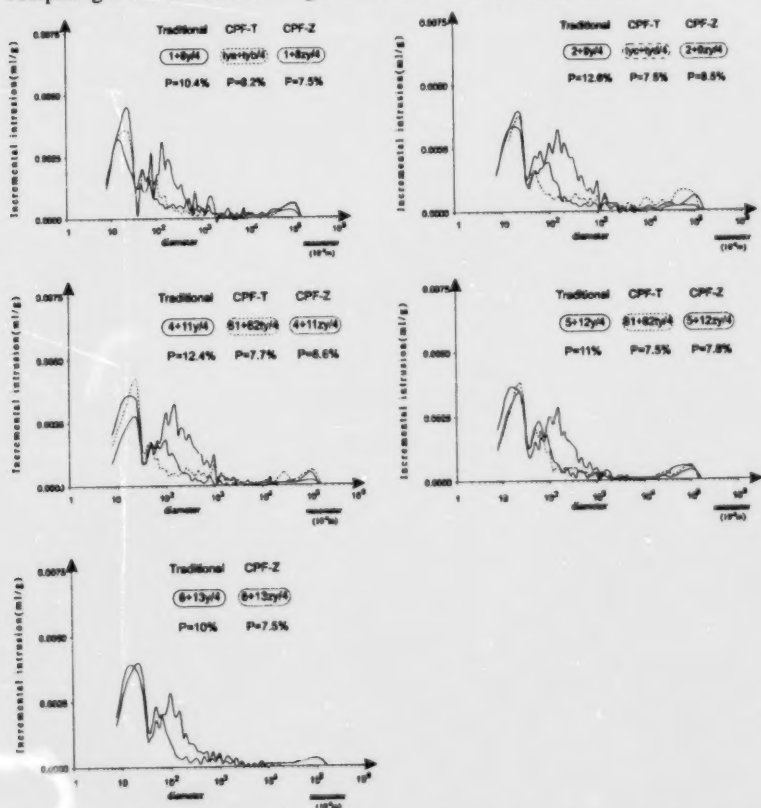


Fig. 7: Some pore size distribution curves for brand 2 top concrete layer for wall C

Comparing in the same manner CPF:T and CPF:Z curves with traditional cast concrete curves (trad.), by analysing Fig. 7 there is also a significant reduction of the quantity of capillary pores.

Porosimetry tests were also carried out deeper down into concrete at depth steps of 4mm, up to a maximum of 20mm. In general, porosity values (TPV) were smaller for concrete cast with CPF compared to the same concrete cast traditionally. Fig.8 shows an example for wall C.

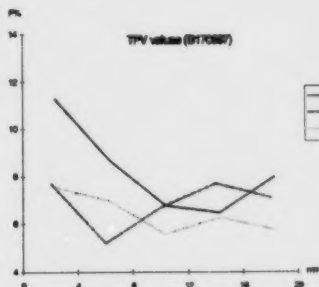


Fig. 8: TPV values versus depth of 4mm slice

3 Conclusions

All results, presented in Table 3, lead to the conclusion that the CPF systems considered do enhance durability mainly in terms of chloride ion penetration and carbonation. Mercury porosity tests confirmed the enormous difference of the pore system of the outer layer of concrete cast traditionally and with CPF systems, thus leading to the conclusion of its beneficial use in works where an optimum quality of the cover to the reinforcement may be crucial in terms of fulfilment of the design life of the structure.

Table 3: Improvement on durability properties comparing concrete cast with CPF systems and the same concrete cast traditionally

Test	Improvement
Surface Hardness	Increase 53 to 102%
Absorption tests	
Initial Absorption (1 hour)	Reduction 59 to 83%
Absorption Coefficient S , 4 hours	Reduction 62 to 89%
Water Penetration depth	Reduction 49 to 95%
Accelerated carbonation	Reduction 66 to 99%
Abrasion tests	Reduction 3 to 14%
Chloride ion penetration	
D Diffusion Coefficient	Reduction 38 to 92%
C_s Surface chloride ion concentration	Reduction 34 to 73%
Porosity (top 4mm layer)	Reduction 32 to 51%

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PART ONE

**DURABILITY OF CONCRETE
MATERIALS AND
STRUCTURES**

INNOVATIVE MATERIALS

3

44 DURABILITY OF HIGH PERFORMANCE CONCRETE CONTAINING PALM OIL FUEL ASH

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Abstract

The use of pozzolans, either naturally occurring or artificially made, has been in practice since the early civilisation. Of the artificial pozzolans, fly ash is the most commonly used globally. With the rise in the demand for high performance concrete in recent years, however, the demand for such pozzolanic materials is also increasing steadily. Perhaps the latest addition to the ash family is palm oil fuel ash (POFA); a waste material obtained on burning palm oil fibre and palm kernel shell as fuel in palm oil mill boilers that has recently been identified as a good pozzolanic material. This paper illustrates some test results on the performance behaviour of concrete incorporating POFA as a partial replacement of ordinary Portland cement (OPC). The durability aspects considered in this study are resistance to heat rise, carbonation, and resistance to aggressive chemicals like chloride and acid attack. Experimental investigation on resistance to heat rise demonstrates that the partial replacement of OPC by POFA is advantageous in controlling temperature rise. As compared with the specimens prepared from OPC alone, concrete specimens made with POFA exhibited much better resistance to attack by chloride ions and acid solution. It is the pozzolanic behaviour of the ash that has been credited for the excellent performance against such aggressive environments. Interestingly, the depth of carbonation was not influenced by the replacement of OPC as only a little difference in each value of carbonation between OPC and POFA concrete specimens has been detected.

Keywords: Carbonation, chemical attack, durability, heat of hydration, high performance concrete, palm oil fuel ash, pozzolan

1 Introduction

The utilisation of supplementary cementing materials, such as fly ash, slags and silica fume has undoubtedly proven the effectiveness of these materials in making strong and durable concrete. With the increase in the demand for concrete with high performance characteristics, the call for such materials is also increasing. Again, with the increase in the diversified use of waste materials with pozzolanic behaviour, more products are likely to emerge as highly potential cement substitute in concrete construction. One of the latest additions to the ash family is palm oil fuel ash (POFA), a waste material obtained in the form of ash on burning palm oil fibre and palm kernel shell as fuel in palm oil mill boilers that has been discovered as a good pozzolanic material.

In recent years, palm oil has become one of the major cash crops in several tropical countries of the world of which Malaysia is the largest producer of palm oil and palm oil by-products. At present, around two hundred palm oil mills are in operation in Malaysia alone where palm fibres and palm kernel shells are used as boiler fuel to produce steam for electricity generation and palm oil extraction process. It has been estimated that the amount of solid waste by-products in the form of fibre and shells generated by palm oil industry is over two million tons per year (Rashid and Rozainee 1993). The ash produced after burning is known as palm oil fuel ash (POFA) and is simply disposed of without any commercial return posing problems of waste management at the mill sites. Laboratory investigations, however, have shown that this ash has pozzolanic properties that not only enabled the replacement of ordinary Portland cement but also found to play an effective role in producing strong and highly durable concrete (Tay 1990; Salihuddin et al. 1996; Awal 1998).

Considering the availability and the inherent pozzolanic property of POFA, research programmes have been initiated in the Department of Structure & Materials of the Universiti Teknologi Malaysia in examining various aspects of strength and durability properties of concrete containing this ash since early nineties. As different pozzolanic materials behave differently, it has been considered felicitous to investigate the effectiveness of this newly identified pozzolanic material in the production of high performance concrete. The durability performances considered in this paper are resistance to heat rise, carbonation, and resistance to aggressive chemicals like chloride and acid attack. Along with durability test data, some physical and chemical properties of the ash are also presented and discussed.

2 Experimental programme

2.1 Materials

2.1.1 Palm oil fuel ash

This ash, as mentioned earlier, is the product of burning palm oil husk and palm kernel shell in palm oil mill boilers. In the present study, ash was collected from Bukit Lawang palm oil mills, Johor of West Malaysia. After collection, the ash was sieved through 300 μm sieve to remove any foreign material and large

size ash particles. The ashes were then ground in a Los Angeles abrasion machine using stainless steel bars (12 mm diameter and 800 mm long) instead of balls in it.

In bulk, POFA is greyish in colour that becomes darker with increasing proportions of unburned carbon. The physical properties and chemical analysis of the ash are shown in Table 1. The most important property that is to mention is the fineness of the ash. Fineness, as measured in specific surface area, shows that POFA is much finer than OPC. Following LeChatelier method, this ash has been found to be equally sound. The chemical analysis reveals that POFA, in general, satisfies the requirements to be pozzolanic and may be classified in between Class C and Class F according to the standard specified in ASTM C618-94a (1994).

2.1.2 Cement

Ordinary Portland cement (ASTM Type I) from a single source was used throughout the study. The detail of the physical properties and chemical composition of the cement is also presented in Table 1.

2.1.3 Aggregate

Dry mining sand with fineness modulus of 2.4 and crushed granite of 10 mm size were used as fine and coarse aggregate respectively. The specific gravity and water absorption of fine aggregates were 2.5 and 1.34 percent respectively while the same for coarse aggregates were 2.61 and 0.76.

Table 1: Physical properties and chemical analysis of OPC and POFA

Tests	OPC	POFA
<u>Physical Properties:</u>		
Fineness - Sp. surface area (m ² /kg)	314	519
Soundness - LeChatelier method (mm)	1	1
Specific gravity	3.28	2.22
<u>Chemical Analysis (%)</u>		
Silicon Dioxide (SiO ₂)	20.2	43.6
Aluminium Oxide (Al ₂ O ₃)	5.7	11.4
Ferric Oxide (Fe ₂ O ₃)	3.0	4.7
Calcium Oxide (CaO)	62.5	8.4
Magnesium Oxide (MgO)	2.6	4.8
Sulphur Trioxide (SO ₃)	1.8	2.8
Sodium Oxide (Na ₂ O)	0.16	0.39
Potassium Oxide (K ₂ O)	0.87	3.5
Loss on Ignition (LOI)	2.7	18.0
Pozzolanic Activity Index with OPC	---	112

2.2 Manufacture of concrete and test details

2.2.1 Design and manufacture of concrete

Concrete with pozzolanic materials can be made following various mix design criteria. In this study, a mix proportion of 1:2:3 for cement, fine aggregate and coarse aggregate, mass for mass, with water-binder ratio of 0.5 was used

throughout the research and the mixing of ash in concrete was based on a simple approach of direct replacement.

Table 2 illustrates the detail of the concrete mix having 30% palm oil fuel ash. One of the important messages of the table is that the total cementitious material content has been kept constant in both the mixes. In order to facilitate the direct comparison of the performance behaviour of POFA with that of the control OPC, no other modifications have been made in the concrete mixes. The only difference in the ash concrete is that 30% OPC amounting to 110 kg/m³ has been replaced by the same amount of POFA.

Table 2: Mix details of concrete with OPC as control and concrete with 30% POFA

Ingredients	OPC Concrete (kg/m ³)	POFA Concrete (kg/m ³)
Ordinary Portland Cement (OPC)	370	260
Palm Oil Fuel Ash (POFA)	---	110
Water	185	185
Fine Aggregate	740	740
Coarse Aggregate	1100	1100

All the tests were performed in the laboratory where the recorded temperature was 27±2 °C with relative humidity of 75±5 %.

2.2.2 Measurement of heat of hydration

Heat of hydration is basically the property of cement concrete in its hardening state. However, the temperature rise due to heat of hydration eventually turns into concern of durability, particularly in mass concrete.

In the present investigation, cubical plywood of sides 500 mm was internally pact with a 76 mm thick expanded polystyrene acting as the insulator. Each concrete mix was cast into another plywood cubical with internal dimension of 300 mm. Prior to casting, a thermocouple (Type K) was inserted into the centre of each box through the drilled hole of the polystyrene foam lid and was connected to a computer driven data acquisition system (Schlumberger SI 3531D).

When concrete was poured into the box, heat was liberated by the hydration process that subsequently increased the temperature of the concrete mass. This increase in temperature and subsequent drop was monitored with close intervals during the first 24 hours and with lesser frequently afterwards until the temperature dropped close to the initial reading. Recording of temperature was continued up to 5 days for both the mixes.

2.2.3 Determination of carbonation

The extent of carbonation of concrete in a particular exposure condition is usually measured in terms of depth of penetration at a certain period. In the present study carbonation tests were carried out on both OPC and POFA concrete prisms (100 x 100 x 500 mm) subjected to different curing conditions, viz. water curing, 7 days in water then air, and air curing. The detail of mix proportions followed in preparing test samples is similar to the one outlined earlier in Table 2.

For each mix and curing condition the carbonation depth was measured at the ages of 28, 90, 180 and 365 days. The extent of carbonation was determined by treating a freshly broken surface of the splitted prism with 1% phenolphthalein; the free calcium hydroxide in uncarbonated portion of concrete turns pink while the carbonated portion remains colourless. The average carbonation for each condition was thus obtained by measuring the distance between the end surface and the colour boundary along the four sides from three specimens.

2.2.4 Determination of chloride penetration

The objective of this test was to evaluate the performance of concrete containing POFA in resisting the penetration of chloride ion. Concrete cylinder specimens (100 x 200 mm) with a mix proportion similar to the one shown in Table 2 were cast. The number of the specimens for both OPC and POFA concrete mixes were nine: three for each immersion period of 7, 28, and 90 days in 5% sodium chloride solution. Day after casting, the specimens were demoulded and cured in water for 28 days before putting them into the chloride solution.

Following the specified period of immersion, the specimens were removed from the solution to test for chloride penetration. At any particular period of immersion, identical cylinders (three OPC concrete and three with POFA) were splitted along their length. The exposed cross-sections were then sprayed with 0.2N silver nitrate solution. Ravindrarajah and Moses (1993) and Lee et al. (1996) have successfully used similar method while determining the depth of penetration of chloride ions.

In the chloride diffused area, silver nitrate forms a white precipitate of silver chloride, while in the area not penetrated by chloride it reacts with hydroxide ions to form silver oxide that appears on the surface as a brown precipitate. The depth of chloride ion penetration in concrete, thus, is clearly identified at the colour change boundary. As the depth of chloride penetration into concrete is not uniform, six readings were taken on each cylinder along the two sides of a split specimen using a digital Vernier calliper.

2.2.5 Test for acid resistance

Various physical and chemical tests on the resistance of concrete to acids have been developed, but there are no standard procedures. Acids of various types, viz. sulphuric acid, hydrochloric acid, acetic acid, nitric acid etc. are normally used in determining the performance of concrete in acidic environment. In the present investigation concrete cube specimens were cured in water for 28 days before putting them into 5% hydrochloric acid solution. The durability performance of both OPC and POFA concrete specimens were then determined by periodic measurement of weight losses of the samples continuously immersed in the test solution. The pH of the solution was controlled to about 2 throughout the immersion period of 1800 hours.

3 Test results and discussion

3.1 Resistance to heat rise

The development of temperature due to heat liberation by the hydration process in both OPC and POFA concrete was determined and the recorded data measured at the mid-depth of concrete blocks are graphically presented in Figure 1. It can be seen that during the initial period the temperature rise for both OPC and POFA concrete was almost equal. With time, a two-fold effect of the partial replacement of OPC by POFA can be detected. Firstly, concrete with POFA reduced the total temperature rise and secondly, it delayed the time at which the peak temperature occurred. Although the initial mixing temperature of both the mixes was approximately the same, considerably more heat was evolved from OPC concrete during the first day i.e. within 24 hours after casting. The peak temperature of 36.7 °C, as obtained for OPC concrete, was recorded at 20 hours. On the other hand, the highest temperature of 35.4 °C was monitored in concrete with 30% POFA at 28 hours of casting. Both the concretes eventually showed a gradual drop in temperature until a relatively steady state was attained during the later period of investigation.

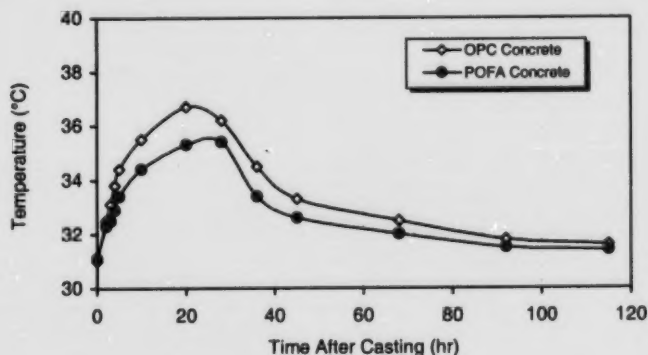


Fig. 1: Effect of palm oil fuel ash on temperature rise in concrete

The observations made above suggest that the partial replacement of OPC by POFA is advantageous for massive structures where potential loss of concrete strength may occur due to thermal cracking. No other experimental data on the time-temperature behaviour of concrete with palm oil fuel ash is yet available in the literature. The available data on other pozzolanic materials like low-calcium fly ash, rice husk ash, silica fume, slag etc. give similar results obtained in this study.

3.2 Carbonation

The carbonation tests were carried out on concrete prisms made from OPC and OPC with 30% POFA. Two objectives were set forward: to evaluate the effect of partial replacement of OPC by POFA and to determine the influence of various curing conditions on carbonation of concrete. The latter was considered

because, apart from the water-binder ratio and compressive strength, curing of concrete containing pozzolanic materials is also known to be a significant factor controlling the rate of carbonation. Data on carbonation of both OPC and POFA concrete obtained at different periods and from various exposure conditions are given in Table 3.

The results presented in Table 3 clearly indicate the importance of initial moist curing against carbonation of concrete. As expected, concretes cured continuously in water showed no traces of carbonation. Both OPC and POFA concrete specimens initially cured for 7 days showed better performance. The depths of carbonation, however, for specimens cured in air were recorded to be maximum at all ages. This is expected, as moist curing provides better hydration; thus improving impermeability of the system resulting from the refinement of pore structure of the mortar phase of concrete.

Table 3: Carbonation properties of OPC and POFA concretes

Curing Regime	Type of Concrete	Depth of carbonation at various test ages			
		28 Days (mm)	90 Days (mm)	180 Days (mm)	365 Days (mm)
Water	OPC	0.0	0.0	0.0	0.0
	POFA	0.0	0.0	0.0	0.0
7 Days water + air	OPC	0.5	1.0	1.5	2.6
	POFA	0.5	0.9	1.6	2.3
Air	OPC	0.6	1.9	3.3	4.4
	POFA	0.8	2.1	3.8	5.0

Although the specimens continuously cured in water showed no carbonation at all, evidently the magnitudes of carbonation depth for OPC and POFA concrete specimens cured in other two conditions were found to increase with time. Data, summarised in Table 3, show some scatter but no remarkable differences had been recorded between the carbonation depths in OPC and POFA concrete specimens.

3.3 Resistance to chloride penetration

Following the procedures outlined in section 2.2.4, the depths of penetration of chloride ion into concrete specimens were measured at different immersion periods of 7, 28, and 90 days. The relationship between the immersion period and the average depth of penetration of chloride ions into concrete with OPC, and concrete with OPC and 30% POFA is shown in Figure 2. The test results, illustrated in the figure reveal that at all times of immersion the depths of penetration of chloride ions into OPC concrete are higher than that in concrete with POFA. This higher resistance to chloride penetration of POFA concrete is apparently due to its pozzolanic behaviour.

Table 4 summarises the chloride diffusion coefficients of OPC and POFA concretes. Because of the sedimentation of chloride ions on the surface of the test specimens, the values of diffusion coefficient for both concretes decreased with

time. However, at all periods of immersion POFA concrete exhibited lower values than the OPC concrete.

No quantitative analysis of chloride content of concrete has been carried out to find a correlation between the chloride concentration and the depth of penetration. However, the result obtained in the present investigation is well in agreement with the findings of Ravindrarajah and Moses (1993) and Collepardi (1993) who found that the depths of penetration of chloride ions in concrete with other pozzolanic materials like fly ash, slag or silica fume were significantly less than in concrete with OPC alone.

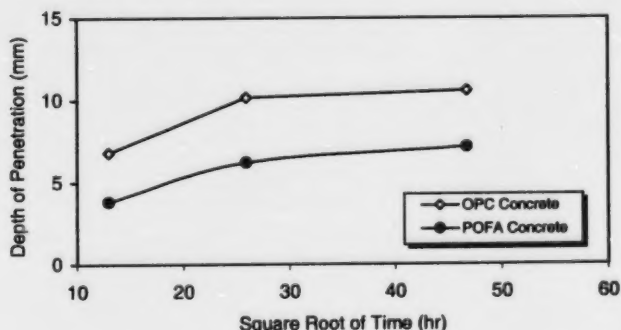


Fig. 2: Effect of palm oil fuel ash on chloride penetration in concrete

Table 4: Chloride diffusion rate in OPC and POFA concrete

Type of Concrete	Diffusion Coefficient ($10^{-3} \text{ mm}^2/\text{hr}$)		
	7 days	28 days	90 days
OPC Concrete	17.25	9.58	3.19
POFA Concrete	5.42	3.44	1.46

3.4 Resistance to acid attack

The resistance to acid attack of concrete cube specimens has been carried out by measuring the loss of weight of the samples continuously submerged in a 5% hydrochloric acid solution. Figure 3 reveals that concrete with POFA exhibited better resistance against the acid at all periods of immersion. The weight loss of OPC concrete after 1800 hours, for example, was 24%; POFA concrete on the other hand lost about 18%. This is expected not only for the fact that POFA is being identified as a good pozzolanic material but also due to its low (i.e. 8.4%) CaO content in comparison to the high content of over 60% in OPC. Because of the higher content of CaO, the hydration products of OPC contain about 25% Ca(OH)_2 which Mehta (1979) concludes to be primarily responsible for the poor resistance of ordinary Portland cement concretes exposed to acidic attack. As POFA contains a very small amount of CaO, consequently the amount of Ca(OH)_2 would be less in the products of hydration. It was also noted that the

test specimens made with POFA showed better surface condition than those prepared with OPC only.

The results obtained in this study are in close agreements with other research findings on rice husk ash and silica fume. According to Mehta (1979) concrete with rice husk ash performed extremely well against various types of acid attack. Experimental investigation by the same researcher with silica fume reveals that concrete exposed to solutions having various percentages of hydrochloric acid, sulphuric acid, lactic acid and acetic acid showed better resistance to chemical attack than the concrete without silica fume.

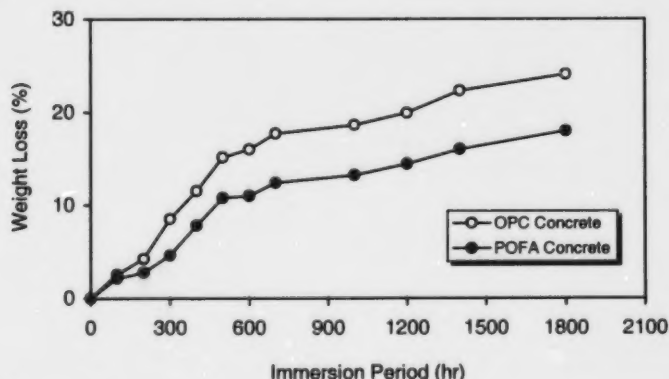


Fig. 3: Comparative weight loss of OPC and POFA concrete specimens continuously immersed in 5% hydrochloric acid solution

The present investigation involves resistance of concrete against only one type of acid solution. Although tests involving other types of acid have not been carried out, concrete incorporating palm oil fuel ash appears to have a potential for the construction of structures exposed to acidic environments.

4 Conclusions

On the basis of the results obtained and observations made in the present study, the conclusions drawn are:

- Experimental investigation on resistance to heat rise demonstrates that the partial replacement of OPC by POFA is advantageous, particularly for mass concrete where thermal cracking due to excessive heat rise is of great concern.
- Generally speaking, concrete with POFA showed somewhat similar depths of carbonation to concrete with OPC alone. Although the results are not truly conclusive, POFA concrete appears to be more sensitive to exposure condition: the dryer the curing condition the deeper the carbonation.

- Study on the resistance to chloride ion revealed that the depths of penetration of chloride ions in concrete containing POFA was much lower than in concrete with OPC alone. The pozzolanic behaviour of the ash causing a decrease in permeability resulting from the refinement of pore structure of the cement matrix has been attributed for the higher resistance to penetration of chloride ions into concrete.
- Like concrete with other fly ashes of low-lime content, POFA concrete exhibited a commendable resistance to acid attack. The pozzolanic behaviour as well as its low lime content had been characterised for the resistance to such acidic environment.

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45 CONCRETE PRODUCTS USING FINE AGGREGATE RECYCLED FROM WASTE CONCRETE PRODUCTS

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Abstract

This paper describes the effective way to use recycled fine aggregate into concrete products. The experiment on the concrete products using fine aggregate recycled from the waste concrete products was carried out. The concrete products were made by curing in a steam to the concrete, compacting in the mold by the strong press and vibration. Rate of recycled fine aggregate to blend with sand was 10, 20, 30, 40, and 50% in a volume in the concrete mix proportion. All concrete specimen was reached to 35MPa target compressive strength. After 300-cycles freezing and thawing test, the durability factor of concrete product specimens using 10-30% recycled fine aggregate blending with sand indicated over 90. Consequently, recycled fine aggregate with poor quality can use up to 30% blending with a good quality sand and a chemical admixture, in case of 0 mm slump concrete products.

Keywords: Concrete products, durability tests (freezing and thawing test), fine aggregate, mechanical tests (compressive and bending tests), recycle, steam curing, waste concrete.

1 Introduction

In a viewpoint of effective utilization of finite resources on the earth, recycling the waste concrete is very important recently. In Japan, they say that a ready-mixed concrete for the construction is amount to about 400 million ton per year, but the volume of waste concrete occurring at the new construction is amount to about 300 million ton, and in the near future it must be much more.

Most of recycle-utilization of the waste concrete used to the paving material of roadbed at the road construction, but a few new concrete constructions. The waste disposal places in the Metropolitan area and environs have been limited and also become very few, and this is a very serious problem in Japan. Furthermore, it

is difficult to use the natural aggregate as a concrete material easily because the mining places to get the natural aggregate decrease actually. Therefore, we must think about reducing the waste concrete volume much more, and about more effective utilization of the waste concrete as the concrete aggregate aggressively. The development of a new recycling technology is now required.

Ministry of construction of Japan, in 1987, developed the rough-standard using the recycled aggregate and recycled concrete for the purpose to recycle as the lower strength-concrete. Actually the recycled aggregate from waste concrete had low quality for concrete materials (Fukushi *et al.*, 1987). The characteristics of recycled aggregate and new recycling machines were introduced (Yamada, 1992). Concerning with a recycling of waste concrete products, Hosokawa (1996) reported that increasing the process crushing waste concrete improved the quality of recycled coarse aggregate. There is certainly a weak point in case of recycling waste concrete into new construction concrete since the recycled aggregate has low quality with cement mortar. Hosokawa (1997) reported the concrete products with recycled coarse aggregate removing cement mortar had good concrete quality compared to crushed stone concrete. Consequently, the recycled coarse aggregate can use as new concrete materials, but recycled fine aggregate is not clear how to use because of their low quality.

This paper describes the effective way of using recycled fine aggregate into concrete products. The experiment on concrete products using fine aggregate recycled from waste concrete products was carried out.

2 Materials and methods

2.1 Manufacturing process of recycled fine aggregate

The original waste concrete was many pieces cut off from the big concrete pipe with the compressive strength of 67.1MPa, which was made with crushed stone. The recycled fine aggregate was manufactured by two crushing process in the crushing plant of Maeta Concrete Products Industries in Sakata city, Yamagata prefecture of Japan. As shown in Fig. 1, the first crush to original waste concrete product was made by the jaw-crusher to the maximum size of 50 mm, and the second crush by the impact-crusher to the maximum size of 15 mm, taking out steel bars at magnet panels and plastic spacers on a way.

The material balance was indicated from original waste concrete products of 100% to 15.5 mm size coarse aggregate of 74.5%, under 5 mm-size fine aggregate of 23.7%, and steel bars and others of 1.8%.

2.2 Concrete materials and mix proportion

The experimental design was shown in Table 1. Concrete materials composed with ordinary Portland cement with 3.16 specific gravity, land and river sands, recycled fine aggregate and crushed stone, and chemical admixture. Aggregate quality was experimented, as shown in Table 2.

The concrete mix proportion of 15 mm maximum gravel size was designed for concrete products with the designed and target compressive strength was 30MPa and 35MPa, respectively, at 26.0% W/C and 0 mm slump shown in Table 3. The concrete product was the cover of gutter of 412x500x95 mm size.

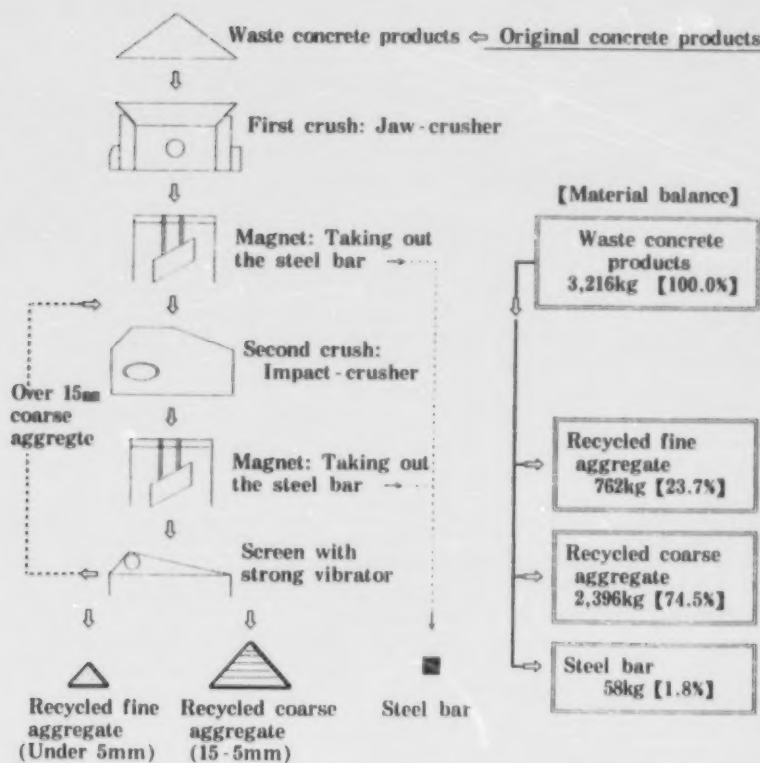


Fig. 1: Manufacturing process of recycled aggregate with material balance

Table 1: Experimental design

Factor	Conditions
Concrete	Water cement ratio (W/C)= 26.0%, Slump = 0 mm. Max. gravel size = 15 mm. Designed compressive strength = 30MPa. Target compressive strength = 35MPa for the Cover of gutter
Cement	Ordinary Portland cement with 3.16 specific gravity
Aggregate	Fine: Mix with land and river sand (30% vs 70% in a weight) Recycled fine aggregate (Rec.) Coarse: Crushed stone
Chemical admixture	Air entraining high-range water-reducing agent: 0.2% of Cement

Table 2: Quality of aggregate

Aggregate	Specific gravity	Water absorption (%)	Finess modulus	Unit Weight (kg/l)	Loss of Washing (%)	Soundness Test (%)	Abrasion Test (%)
Land sand	2.56	2.25	1.72	1.57	1.30	1.18	-
River sand	2.56	2.22	2.99	1.57	1.40	3.76	-
Rec.	2.14	13.30	4.19	1.44	3.70	10.95	-
Crushed stone	2.62	2.02	6.39	1.51	0.30	3.73	18.40

Table 3: Concrete mix proportion

Concrete	W/C (%)	s/a (%)	Water	Cement	Unit weight (kg/m ³)			Gravel	Chemical mixture
					Sand	River	Rec.		
					Land				
P	26.0	53.0	100.0	385.0	295.0	695.0	0.0	897.0	0.000
0 *	26.0	53.0	100.0	385.0	295.0	695.0	0.0	897.0	0.770
10 *	26.0	53.8	101.2	389.2	269.8	627.2	83.5	878.3	0.778
20 *	26.0	54.6	102.4	393.8	242.5	563.7	169.0	860.0	0.787
30 *	26.0	55.4	103.6	398.5	214.5	498.6	256.2	841.7	0.797
40 *	26.0	56.2	104.8	403.1	185.8	431.9	345.2	823.5	0.806
50 *	26.0	57.0	106.0	407.7	156.5	363.7	436.0	805.5	0.815

* No. is indicated the percentage of blended Rec. with land and river sand.

2.3 Methods of manufacturing and curing concrete product and specimens

The fresh concrete for concrete product with 19.6 lit. was mixed by a 100 lit. forced mixing pan-type mixer and filled up into the mold without steel bars for specimens of the compressive and bending strength and the freezing and thawing test, and with steel bars for the breaking load test only. Compacting by the machine with 188MPa-press and 3,600 rpm-vibration, the concrete products were carried in the steam curing room, and they were kept for 2-hours 60°C top-temperature as shown in Fig. 2. After removing molds, each specimens were cut off by a diamond-wire saw to 412×100×95 mm size and by a 50 mm diameter-core boring-machine from the concrete products without steel bars, after 7-days field or standard curing as shown in Fig. 3.

In each concrete test, the result was indicated by the mean of three specimen values because of using three specimens. All specimens was measured the unit weight and the dynamic modulus of elasticity.

2.4 Tests of compressive and bending strength, and freezing and thawing test

The compressive strength of standard curing specimens were measured at 28 days, and the compressive and bending strength of field curing one and the breaking load of concrete products with steel bars were measured at 14 days.

The freezing and thawing test was started at 14 days of field curing specimens, after the cutting surfaces were painted an epoxy resin. One cycle of freezing and thawing test needed about 4 hours, and the temperature of specimen's center was changed 18 to -5°C with freezing and thawing in a water, based on JSCE (1986) method. The dynamic modulus of elasticity and weight of specimens were periodically measured in each 30 cycles until 300 cycles, the durability factor and the weight loss of each specimen were calculated.

Furthermore, the air-content, i.e., entrained air and total air void, in hardened concrete specimens were measured by a microscope, based on ASTM C-457.

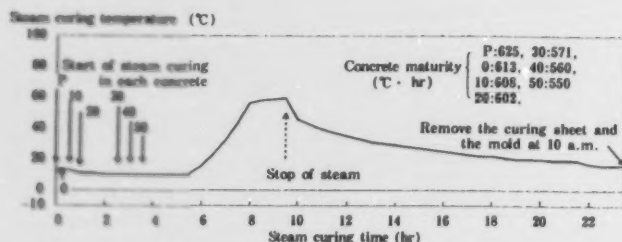


Fig. 2: History of steam curing temperature and concrete maturity

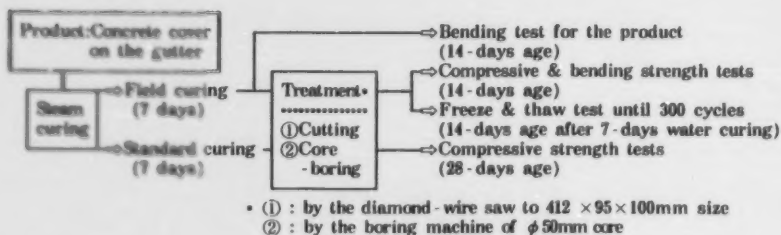


Fig. 3: Treatment and test for the specimens after steam curing

3 Results and discussion

3.1 Aggregate quality, fresh concrete properties and steam curing condition

As shown in Table 2, crushed stone and land and river sands had good quality. However, recycled fine aggregate indicated very poor quality with specific gravity of 2.14 and percentage of water absorption of 13.30%. If using recycled fine aggregate into the concrete, the concrete strength and durability may show a poor result, and recycled fine aggregate should blend with good quality sand while confirming in the experiment.

Fresh concrete slump was 0 mm, being a very hard fresh concrete, and the quality of concrete was checked by the skilled worker if the concrete was suitable or not for the products. The contents of surface water of sand may control the concrete quality. Fresh concrete air could not measure because it's too hard.

The history of steam curing temperature and concrete maturity was shown in Fig. 2. Since the time to finish the mixing concrete was different each concrete, the setting time before starting a steam curing ranged from 2 to 6 hours. The specimens were removed molds at 10 a.m. on the next day. Therefore, the maturity, calculated as the base line of -10°C after mixing concrete to removing the molds, ranged from 550°Chr to 625°Chr . These differences among 7 maturities were small, and it is estimated that the difference's influence in the concrete strength is small.

3.2 Compressive and bending strength of specimens and breaking load of product

Fig. 4 shows the compressive strength of each concrete. All of specimens had over 35MPa target compressive strength. The field curing strength indicated the same tendency of the standard curing strength. The 10% recycle fine aggregate blending concrete had the highest strength than other concrete. Consequently, the recycled fine aggregate blending up to 50% with sand had no problem in the concrete compressive strength. The correlation coefficient between concrete maturity and compressive strength was 0.538 with less correlation because of narrow range in the compressive strength. Also, the correlation coefficients between dynamic modulus of elasticity and compressive strength were 0.089 in field curing and 0.285 in standard curing with both no correlations. The correlation coefficients between unit weight and compressive strength were 0.356 in field curing and 0.348 in standard curing, and there were low correlations ($p < 0.05$).

Fig. 5 shows the ratio of bending strength to compressive strength. The ratio of bending strength to compressive strength distributes generally from 1/5 to 1/7. In the current study, this ratio distributed from 1/7 to 1/8, indicating a slightly smaller bending strength compared to general concrete, and the lower ratio was found in 40% and 50% blending especially.

The breaking load test of the concrete product "Cover of gutter" is a similar test method with a bending strength. Their correlation coefficient was 0.108 with no correlation, and the breaking load of concrete product with steel bars distributed the narrow range of 90-120kN. In the concrete mix proportion, higher the blending rate of recycled fine aggregate, the more the cement quantity, like 407.7 kg/m^3 in 50% blending. As every concrete had over 35MPa compressive

strength, the way using a recycle fine aggregate with lower quality is effective in case of 0 mm slump concrete products with 26% W/C.

Compressive strength (MPa)

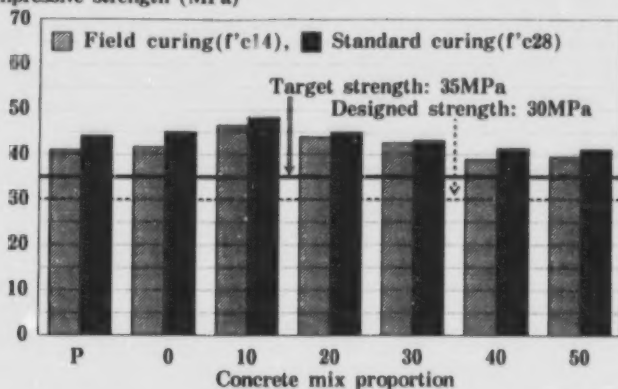


Fig. 4: Compressive strength

Bending strength [f_b] (MPa)

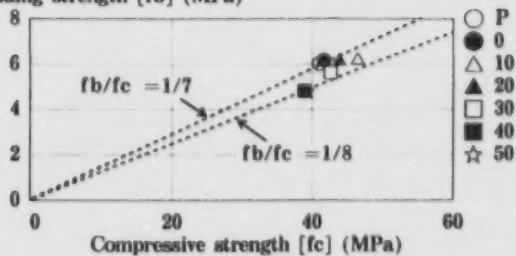


Fig. 5: Ratio of bending strength to compressive strength

3.3 Freezing and thawing durability of concrete specimens

Table 4 shows the durability factor and weight loss of concrete specimens as results of freezing and thawing test. After 300-cycles test, the durability factor indicated over 95 in the concrete of 0, 10, and 20% blending, and ranged 90-95 in the concrete of P and 30% blending, but under 90 in the concrete of 40 and 50% blending. Furthermore, the specimen's weight loss indicated over 1% in the concrete of P and 50% blending with a scaling of specimens surface, and less weight loss in the concrete was 0, 10, 20, and 30% blending.

Table 4: Results of freezing and thawing test and entraining air in hardened concrete

Concrete	Durability factor	Weight loss (%)	Entraining air (mean)		Water absorption (%) *	f'c14 (MPa)
			Air (%)	Spacing factor (μm)		
P	91.1	1.11**	0.21	446	2.13	41.0
0	95.0	-0.05	0.59	168	2.13	41.7
10	96.5	0.12	0.64	174	2.73	46.5
20	97.8	-0.18	0.76	200	3.34	43.9
30	91.9	-0.06	0.94	209	3.97	42.5
40	81.3	0.08**	1.04	198	4.62	38.9
50	55.6	2.58**	0.59	238	5.29	39.4
MRA***	Y		X1		X2	X3

*In the total aggregate.

**Surface scaling of specimens.

*** Formula, multiple correlation coefficient (R) and partial regression coefficients (Rx1, Rx2 and Rx3) by a multiple regression analysis are;

$$Y = 63.7 + 33.1x_1 - 12.6x_2 + 1.05x_3, R = 0.966, Rx_1 = 0.875, \\ Rx_2 = -0.937, Rx_3 = 0.507.$$

The total air void in hardened concrete had no correlation between the compressive strength and the space between air voids, as shown in Fig. 6. However, in Table 4, the concrete P had 0.21% entrained air and wider spacing-factor 446μm without chemical admixture, and the durability factor had good correlation with entrained air and less water absorption in the total aggregate, by a multiple regression analysis. The water absorption in total of aggregate, can calculate from each water absorption of aggregate, must be reduced for the good durability, but it's very difficult.

Although the general concrete with 45-55% W/C will not be durable in the freezing and thawing test, the durability in concrete products was caused by smaller W/C of 26% and higher compressive strength of over 35MPa, in spite of using low quality recycled fine aggregate. Consequently, in the view of concrete durability, recycled fine aggregate can use up to 30% blending with a good quality sand and a chemical admixture in case of 0 mm slump concrete products.

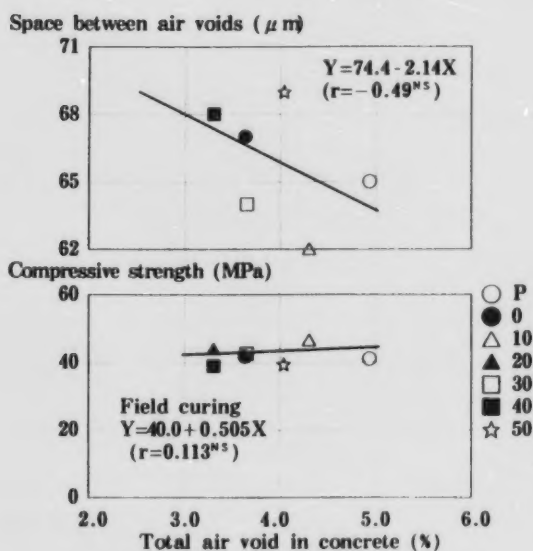


Fig. 6: Relationship between total air void in concrete and the compressive strength and the space between air voids

4 Conclusions

Major findings of this study include the following:

1. Recycled fine aggregate indicated a very poor quality with specific gravity of 2.14 and percentage of water absorption of 13.30%. In case of using them as concrete materials, they should blend with good quality sand.
2. The concrete products "Cover of gutter" were made with recycled fine aggregate at the condition of 26% W/C, 0 mm slump and about 400 kg/m³ cement, and cured in a steam after compacting by the strong press and vibration. 6 kinds of experimental concrete were included P without recycled fine aggregate and chemical admixture, and 0, 10, 20, 30, 40, and 50% recycled fine aggregate blending with sand and chemical admixture.
3. All concrete specimens of 14-day field curing and 28-day standard curing indicated over 35MPa target compressive strength.
4. After 300 cycles freezing and thawing test, the durability factor indicated over 95 in the concrete products of 0, 10, and 20% recycled fine aggregate blending, and ranged 90-95 in the concrete of P and 30% one blending, but under 90 in the concrete of 40 and 50% one blending. The durability factor had a good correlation with entrained air and less water absorption in the total aggregate.

5. Recycled fine aggregate with poor quality can use up to 30% blending with a good quality sand and a chemical admixture, in case of 0 mm slump concrete products with 26% W/C and 30MPa designed compressive strength.

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46 DURABILITY OF POLYMER MODIFIED AND IMPREGNATED GYPSUM

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Abstract

The experimental study was carried out in two stages: Stage one constituted a test of the mechanical strength and moisture resistance of hemi-hydrate gypsum ($\text{CaSO}_4 \cdot \frac{1}{2}\text{H}_2\text{O}$) combined separately with acrylic, epoxy and silicone resins. With the objective of determining the best-performing mixture, different types of acrylic, silicon and epoxy-based polymers in varying quantities were combined with gypsum. Succeeding the capillary trials of the gypsum specimens, the specimens were subjected to submersion in water for 7 and then 15 days. The specimens were then weighed to establish the relative amount of moisture adsorption. Measurement of the mechanical strength of the specimens was conducted by compressive- and flexural tests.

Stage Two focused on tests of the moisture durability of gypsum compounds that had been impregnated with acrylic and epoxy emulsions. The degree of polymer impregnation of the specimens was determined. Subsequent to the capillary tests, the specimens were submerged in water for 7 days. The level of water absorption was ascertained, and the depth of the polymer impregnation attained in the specimens was calculated.

The findings of this study revealed that the best performance among the different resins was displayed by the specimens containing silicone resin. Though the moisture durability of gypsum compounded with silicone resin was less than 100%, it was demonstrated that gypsum modified in this manner can be utilized as an outdoor material. The next best results were offered by modification with epoxy resin. The outcome of the impregnation tests also indicated that silicone resins yielded the greatest durability to water adsorption and that epoxy resins came second.

Keywords: Gypsum, silicone resins, epoxy resins, acrylic resins

1 Introduction

In use for more than 5,000 years, gypsum is still reserved a place among indispensable building materials on account of its outstanding properties. Among the positive qualities of gypsum valued in the construction business are its good thermal and sound insulation, light-weight, ease of application, resistance to fire, hydrometric dimensional stability, smooth surface, good compressive strength, and neutral pH.

Further, it is a readily available natural resource. It should be also noted, that the raw material gypsum is increasingly being replaced by technical gypsum which is a waste product of industry. The most common type of such materials is Reagypsum, an industrial by-product of power station plants that undergoes a post-recovery purifying process.

The main defect of gypsum is its lack of resistance to moisture, making it unsuitable for use on exposed exteriors. For this reason, much research on gypsum has been aimed at enhancing its resistance to atmospheric conditions. In this connection, the present study on gypsum was intended to discover a means for overcoming its poor moisture resistance, so as to permit its utilization in exterior construction elements. Correlation was made in the current study of the relative efficiency of the polymer-based additive substances of acrylic, silicone and epoxy resins in both modified gypsum compounds and superficially impregnated gypsum blocks.

This experimental investigation was carried out in two stages: Stage one constituted a test of the mechanical strength and moisture resistance of gypsum ($\text{CaSO}_4 \cdot 2\text{H}_2\text{O}$) combined separately with acrylic, epoxy and silicone resins. With the objective of determining the best performing mixture, different types of acrylic, silicon and epoxy-based polymers in varying quantities were combined with gypsum. Succeeding the capillary trials of the gypsum specimens were subjected to submersion in water for 7 days. The specimens were then weighed to establish the relative amount water adsorption. Measurement of the mechanical strength of the specimens was conducted by compressive- and flexural- strength tests.

Stage Two focused on tests of the durability of gypsum material against water that had been impregnated with acrylic and epoxy emulsions. The degree of polymer impregnation of the specimens was determined. Subsequent to the capillary tests, the specimens were submerged in water for 7 and then 15 days. The level of water absorption was ascertained, and the depth of the polymer impregnation attained in the specimens was observed.

2 Materials and method

The specimens were prepared with the aim of measuring the degree of moisture resistance obtainable by gypsum material that had undergone modification through the addition of epoxy, silicone and acrylic resins (for the proportions, see Table 1). Each test run was conducted with 10 specimens measuring 4 x 4 x 6 cm, of which 6 specimens each were subjected to capillary and water absorption and density tests and 4 each were employed in flexural and compression strength trials. In addition, for purposes of comparison, another 10

specimens were prepared of unmodified gypsum, each measuring 4 x 4 x 16 cm, all of which were dried for 3 days in an oven by 40°C to yield a measure of their dry weight. These specimens were weighed again after an interval of 24 hours of exposure to 65% humidity and at a temperature of 20° C ± 2°C to determine their weight under conditions of atmospheric humidity. Based on the results of the preliminary trials for consistency determination, the composition of the experimental specimens of gypsum combined with silicone and epoxy resins possessed a 60% ratio of water/gypsum (60 gr. water to 100 gr gypsum). But, due to the increased water demand of the acrylic resins at 60%, the gypsum compound became too dry for production of the specimens with acrylic resins and were prepared with an 80% water ratio. Only distilled water was used in the preparation of the specimens and the trials on capillarity and water adsorption.

Table 1: Matrix of prepared specimens from each resin type and ratios

	0,5%	2,5%	3%	5%	8%	10%	13%	43%
Silicone	10	10	-	10	-	10	-	-
Epoxy	-	-	10	10	-	10	-	-
Acrylic	-	-	-	-	10	-	10	10

Table 2: Resin types used in the experiments

Base Type	Firm and Label	Application
Silicon	Wacker BS46	Modifying
Epoxy	Araldite	Modifying
Acrylic	STT	Modifying
Silicon	Wacker 290	Impregnation
Epoxy	Aradite, Epoxy Grout S	Impregnation
Acrylic	Heydi, Impregnation Fluid	Impregnation

The proportion of the additive resins used in the specimens was calculated as part of the water used in the mixture and a corresponding deduction was made in the amount of water employed. As the test program for the investigation of the durability of gypsum material with in table 1 mentioned additives, the Volume weight, Flexural- and Compression strength tests, Capillarity tests and Water Adsorption tests are conducted.

3 Experimental method and results

This section contains a description of the procedure and a presentation and evaluation of the results.

3.1 Capillarity

The specimens, whose weight reflected conditions of atmospheric humidity, were placed on a grill with 1 cm thickness in a tray in such a manner that the undersurface of the specimens were in contact with the water and weighed at regular intervals. The weight values obtained were used in the formulas given below to determine the water adsorption ratio by capillarity.

$$\text{Capillarity } E = \frac{Q}{A} \text{ cm}^3/\text{cm}^2 \quad N = \frac{\Delta E}{\sqrt{t}} \text{ cm}/\sqrt{t}$$

As the water was absorbed by the specimens in the capillarity trials, the water in the tray was replenished and the degree of water absorption was measured at minutes 4, 9, 16, 49, 64, 81, 100, 121 and 149 th.

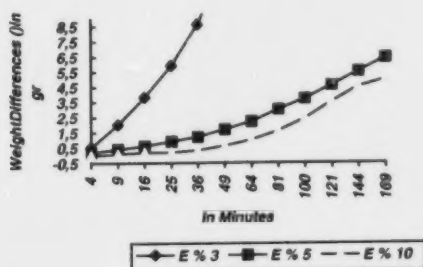


Fig. 1: Capillarity of epoxy resins

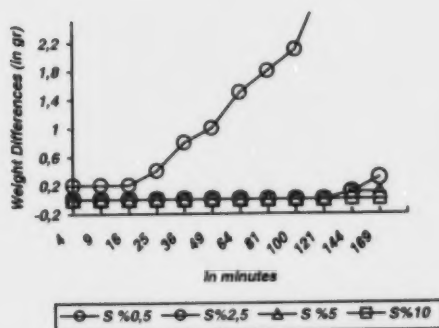


Fig. 2: Capillarity of silicone resins

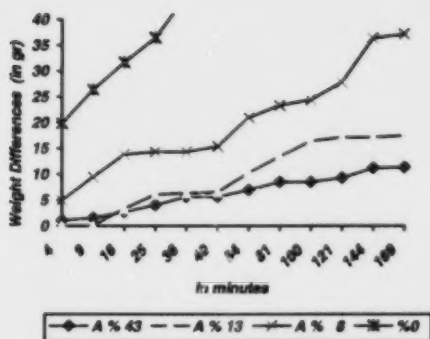


Fig. 3: Capillary of acrylic resins

- In the capillary tests, the specimens modified by silicone resins demonstrated an undeniable superiority of durability against moisture by capillary action. In relative terms, the least amount of durability to exposure to moisture exhibited by silicone that is, at a ratio of 0.5% (of the amount of to the water which is added to the gypsum) can be compared with the greatest degree of durability shown by epoxy resin, which was obtained only at the much greater ratio of 10%. Though an increase in the ratio of silicone tended to produce a corresponding increase in moisture resistance, little gain in resistance was procured by an increase from 5% to 10%. The addition of 0.5% silicone to the specimens resulted in also a better performance than a gypsum compound containing 43% acrylic resin.
- As can be seen in the graphs of the capillary trials, the epoxy resins exhibited a greater degree of durability against moisture than unmodified gypsum and acrylic resins.
- In the capillarity tests, the acrylic resins displayed the least moisture resistance among the three kinds of resins.
- The existence of a slight 0.1% gain in weight in the gypsum compounds with 10% silicone resin in the capillarity trials can be regarded simply as a signifier of surface moisture.

3.4 Water adsorption

The specimens for the capillarity trials were first placed in a tray in which water was added so that it would successively reach a level of one-third, two-thirds and three-thirds, relative to the height of the specimens. This was designed to drive out any air contained in the gypsum. The specimens were weighed after their submersion in water for 2 days, which constituted the saturated weight for each. The oven-dry weight was deducted from the saturated weight of each sample to calculate the amount of water that had been absorbed.

From the results of the water absorption trials can be maintained that;

- The specimens with silicone resin gave similar to the capillarity tests the best results. Water adsorption trials with epoxy resin served to place this gypsum compound in the second place with the least resistance to moisture. While testing better than unmodified gypsum, epoxy resin endowed gypsum with no great success. Another factor revealed in the test results of epoxy-modified gypsum was a the combining of the epoxy resin with the water in the gypsum, which caused a foaming in the water while stirring and therefore porosity in the gypsum compound.
- Though acrylic resins proved the least moisture resistant in the capillarity tests, the one with 43% ratio, exhibited a better performance than epoxy resins in water adsorption tests. This is undoubtedly due to the fact that the proportion of acrylic resin was greater than (which had to be high proportion according to the manufacturer's advise) those of the other two resins. The composition ratios of 43% of acrylic resin to the water added to the gypsum can be compared, for instance, with 10% at the most for epoxy and silicone resins.

3.3 Volume weight

The measuring of the volume weights of the specimens were included to the experimental program in order to determine the porosity of the specimens as the water adsorption of a material has a directly relationship with its porosity unless the cellular structure is in a closed form.

For the determination of the volume weights, the specimens were subjected to the water adsorption trial. These specimens were weighed in a vessel of water suspended from the scale to determine their weight in water. The obtained values from these weighings were divided into the oven-dry weights of the specimens to establish the unit volume weights.

Table 3: Determination of the volume weights

Types of Resin	Matrix %	Average Volume Weight gr/cm ³	Number tested	Average Capillary coefficients cm ³ /v min.	Average Water Adsorption Ratio
Silicone	0,5	2,55	4	0,010054	0,232
	2,5	4,33	4	0,000081	0,082
	5	5,16	4	0,000080	0,034
	10	5,47	4	0,000074	0,017
Epoxy	3	2,30	4	0,102697	0,27
	5	2,33	4	0,017082	0,264
	10	2,35	4	0,009753	0,250
Acrylic	8	2,63	4	0,1562718	0,292
	13	3,23	4	0,068561	0,241
	43	3,42	4	0,048473	0,179
Without Additive	0	2,14	4	0,4559122	0,308

3.4 Mechanical strengths

Mechanical and compression strength tests were conducted to establish the mechanical strength of the gypsum specimens containing the various proportions of silicone, epoxy and acrylic resins (see Table 1).

3.4.1 Flexural strength

Measuring 4 x 4 x 16 cm, the flexural strength of the specimens of gypsum combined with the resins in the proportions shown in Table 1 was tested by crushing them in the manner indicated below with a fracture rate of 2 N/mm². In determining the flexural strength, the full width and length of the specimens were measured and formulated. The results of the flexural strength test are presented in Table 4.

$$\tau = 150 \times \frac{P}{bh^2} \text{ N/mm}^2 \quad P = \text{Force in Newton} \quad b = \text{width} \quad h = \text{height in mm}$$

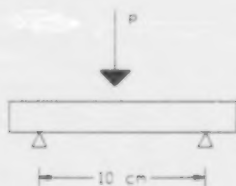


Fig. 4: Performing of flexural strength tests

3.4.2 Compression strength:

To calculate the compression strength of the specimens, they were crushed by the application of pressure to their upper surfaces with a head measuring 4 cm. The strength values obtained were inserted in the compression strength formula indicated below. The compression strength values of the specimens are given in Table 4.

$$\sigma = \frac{P}{A} \text{ N/mm}^2 \quad \sigma = \text{Compression strength} \quad P = \text{Force} \quad A = \text{Area in mm}^2$$

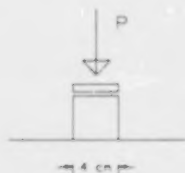


Fig. 5: Performing of compression strength tests

Table 4: Mechanical strength of the resin modified gypsum specimens

Resin Types	Resin matrixes %	Compressive Strength MPa	Flexural Strength MPa	Volume Weight g/cm ³
Silicone	0.5	14,21	5,63	2,55
	2,5	16,07	5,78	4,33
	5	16,51	6,03	5,16
	10	16,99	6,04	5,47
Epoxy	3	12,88	5,40	2,30
	5	12,39	6,46	2,33
	10	14,35	6,44	2,35
Acrylic	8	9,95	3,84	2,63
	13	18,51	6,44	3,23
	43	20,65	7,24	3,42
Without Additive	0	14,83	5,60	2,14

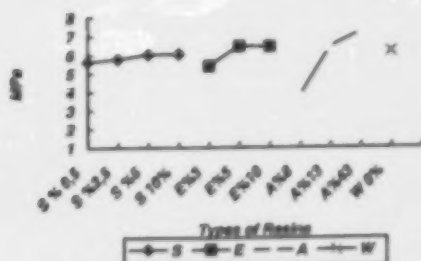


Fig. 6: Flexural strength of the test samples

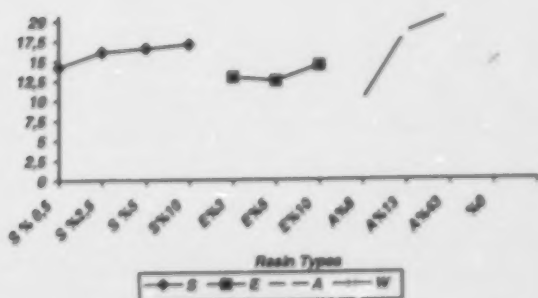


Fig. 7: Compression strenghts of the test samples (in average)

3.4.3 Results of the mechanical strength trials

- Among the specimens compounded with resins in the study, those with the proportion of 43% acrylic resin demonstrated the highest flexure and compression strength. Second best was again acrylic resin in the ratio of 13%. The fact that the specimens with these two ratios of acrylic resin exhibited high performance in both compression and flexure strength can largely be attributed to the excessive amount of resin added.
- Second place in compressive strength was occupied by silicone resins.
- Specimens with 5% and 10% epoxy resin components possessed greater flexural strength than the specimens with silicone resins.
- In the compression and flexural strength tests, the lowest degree of mechanical strength was displayed by specimens with 8% acrylic resin.
- The compression and mechanical strengths of the unmodified gypsum specimens was approximately the same as those with 0.5% silicone resins which is very natural because of the low ratio of the resin. But, the addition of 0.5 % silicone resin endowed the gypsum material with a much greater resistance to moisture.

3.5 Impregnation

A layer of silicone-, epoxy- or acrylic-based resin was applied by brush to the surface of each of the specimens of unmodified gypsum, after which the specimens were left to dry. When the resins were cured in laboratory conditions (20°C, 65% humidity) for 7 days, the specimens underwent capillary and water adsorption tests. The amount of protection product absorbed by the specimens was determined by calculating the difference between dry weights before and after treatment. The weighings are made 7 days after the application of the product and repeated 7 days later. The average amount of protective product absorbed by the specimens in various times is given in table 5. The weight of the product absorbed generally tends to diminish in time because of their progressive polymerisation. This behaviour is stressed for silicone treated specimens which present loses of weight reaching up to 50 % after 15 days. In the case of epoxy treated specimens, the water evaporates very rapidly due to the high porosity of gypsum specimens and the product weights are stabilised in a short time. Whereas the specimens treated with acrylic show moderate difference of weight probably caused by its slower rate of polymerisation.

The specimens treated with epoxy and silicon show the same trend. The amount of water absorbed by these specimens remains very small in time assuming a high and lasting protection. The protection furnished by silicone is always excellent even if the amount of protective absorbed on specimens is often very different. These results indicates that smaller protective amounts are sufficient to assure good protection.

The different trend of capillarity curves for the two classes of products is very significant. Silicon products show straight lines very close to the abscissas axis, whereas acrylic products show curves similar to those obtained for non-threatened specimens, though a reduction, sometimes high of water absorbed by capillarity is observed. Data obtained seen therefore to confirm the hypothesis that silicone products form on the sample a sort of continuous and resistant protective layer. On the other hand, acrylic products seem to act in a different

manner. They slow down the speed of water entry, but do not form a barrier able to prevent its penetration. In other words, silicon based products arrange mostly on the surface of the sample while acrylic ones penetrate more deeply filling the pores and then diminishing the available volume.

Table 5: The amount of protective product absorbed by the specimens

Protection Type	Amount of Protection Product	
	7 days	15 days
Silicon	0,50	0,25
	0,40	0,22
Epoxy Emulsion	0,53	0,53
	0,44	0,43
Acrylic	0,93	0,90
	0,99	0,95

* q of protective absorbed for 100g of gypsum sample

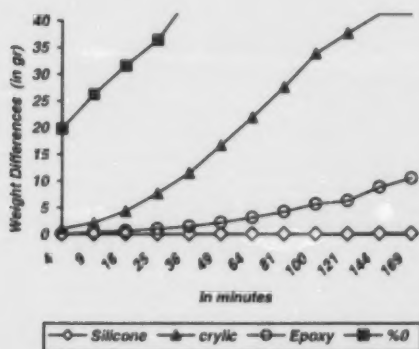


Fig. 8: Capillarity of impregnated specimens

4 Conclusions

The findings of this experimental investigation of the durability of modified and impregnated gypsum by silicone, epoxy and acrylic resins are summarised as follows:

- In the capillary trials, the specimens modified by silicone resins demonstrated an undeniable superiority of durability against moisture by capillary action. The second good results revealed the epoxy resins.
- As in the capillary trials, the silicone resins also showed superiority over the other two resin types in the water adsorption trials. An increase in the proportion of silicone resins in the gypsum specimens produced a direct

correspondence in its level of moisture resistance. For instance, the addition of a 10%-ratio of silicone resin to a test sample of gypsum produced a 17-times higher durability level compared with unmodified gypsum. Compared with other materials, such as cement and brick, the low level of water adsorption of gypsum with silicone resin makes it suitable for exterior construction use. Though, no gypsum compound was obtained that possessed 100% moisture resistance.

- The unit volume weight of silicone resins was greater than that of the other two resins. This indicates that silicone resin fills the spaces between the gypsum molecules and decreases its porosity. Thus, silicone resin not only possesses a water-repellant character but it endows the material from itself with the quality of moisture resistance.
- Because the specimens with acrylic resin content represented the large proportions of 13% and 43%, their unit volume weight showed an increase. Nevertheless, this was insufficient for a more than satisfactory performance in moisture durability.
- Moreover, in the tests designed to provide gypsum with moisture durability by the impregnation method as well as by other methods, such as spreading the resin over the surface, produced results similar to that with modified gypsum. In the impregnation system, also, silicone resins produced the best results in both the capillarity and water adsorption tests. In second place were the epoxy resins and in the specimens with resin-covered surfaces the least satisfactory performance was displayed by the acrylic resins.

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47 THE PHYSICAL PROPERTIES OF POLYSTYRENE AGGREGATED GYPSUM BLOCKS

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Abstract

Lightweight concrete is widely used in our day as a filling material for sandwich panels in the construction industry. On the other hand, the use of gypsum in this aspect is very limited. Block or panels as partition walls produced by foamed or porous gypsum has not been widely publicised. According to literature, similar types of gypsum panels are produced and used in Israel and the former East Germany. During our literature survey, using polystyrene beads as an aggregate for the production of the gypsum blocks or panels has not been discovered. In this study, we have observed the use and production gypsum composites, using polystyrene beads. Even though it is possible to mix polystyrene beads and gypsum without additives, better results are obtained by wetting the beads using a glue with a polymer base for binding as the mixture become more homogeneous. Production is planned for samples between 200kg/m^3 and 800kg/m^3 density, among those $500\text{--}600\text{ kg/m}^3$ mixtures have been found to be the most economical ones and the cavities filled with gypsum up to the maximum extent. Even though the mechanical strength is not a critical issue for these samples, it is found to be sufficient, in addition good heat insulation and fire retardant properties has been observed.

Due to the above mentioned properties, polystyrene aggregated gypsum can be used in precast or cast in situ panels as a filling material.

Keywords: Gypsum, polystyrene foam, epoxy resins

1 Introduction

Lightweight concrete is widely used in thermal insulation, to decrease the cross sections of reinforced concrete structural elements or in block or panel systems. But use of light-weight gypsum as a filling agent or as prefabricated

panels is very rare. For example of these applications is the production of hollowed gypsum blocks for use as domestic partition walls. The application of making gypsum light by foaming is also scarce, and such productions do not permit the material to have a density less than 600kg/m^3 . Productions made with light aggregates like perlite and vermiculite are not so effective, due to their small ratios in the mixture.

Using polystyrene foams as light aggregate is quite new. Polystyrene foams produced by the German firm BASF, and sold under the trade name of Styropor are given to the manufacturers as Styropor beads. The manufacturers put these beads in suitable moulds and heat it with steam in order to give them the shape of the mould. Experiences in using half-expanded 1.5-2 mm spherical beads as aggregate material in concrete, were tested since 1949. Although the experiments were highly successful, the application of this practice was not developed, since that condition of polystyrene foam was very expensive at that date. However when the pre-expanded polystyrene foam beads became economical after 1960's, polystyrene foam cement found a wide enough application field.

Presently, the polystyrene foam finds a wide application as insulation material in board form. It is also widely used in packing. In companies that use polystyrene foam beads by expanding the beads, there is a lot of waste material left because of large dimensions and faulty productions. These left overs are simply crumbled and turned back to polystyrene foam beads. A firm dealing with polystyrene bead production, consulted Istanbul Technical University Faculty of Architecture's Construction Materials Laboratory and wanted to evaluate these leftovers. In the performed research, among other findings, it was understood that these beads could be used together with gypsum, just like in cement to form gypsum with polystyrene foam.

2 Definition

The production of gypsum productions with polystyrene foam aggregate give various advantages with respect to other gypsum mixtures. First of all, crumbled surplus polystyrene foam beads can be purchased after production for a very low price, and very light products are resulted. Polystyrene foam beads can be used as volume 50-70% depending on the density of the gypsum block. By using crumbled polystyrene foam beads the following can be accomplished:

- Very small observed unit volume weight ($\Delta=11-12\text{kg/m}^3$)
- Very small heat transfer value ($\lambda=0.04\text{ W/m }^\circ\text{C}$)
- Durability against moisture penetration due to the closed cellular structure of the beads.
- Due to formation of the beads close to each other, the remaining cavities between them are filled tightly with matrix material which gives a high mechanical strength to the gypsum composite.
- Since the form of the polystyrene foam beads are perfectly spherical, they distribute themselves evenly in concrete or gypsum paste so that construction elements in homogeneous structure can be produced.

3 Material

3.1 Polystyrene foam beads used as aggregate

The basic material is expanded Polystyrene foam, produced by the German firm B.A.S.F. and being exported to the world under the name of Styropor since 1952. Manufactured as board and sandwich panels, it is widely used in construction, insulating and packaging industries.

The manufacturing of the material is realised in two stages. Stage one; the unexpanded material called polystyrene foam bead is pre-expanded. Stage two; these beads are closed in forms and fused together. At this moment the beads touch and are welded to each other in the contacting boundaries and reach their final form as blocks, boards or shaped mouldings.

The raw material consists of fine cellular beaded polystyrene which is subjected to a pre-expansion process. In this process, the fine polystyrene beads containing an expanding agent are heated in steam, the expanding agent in the fine beads vaporises and the vapour of the agent mixed with steam expands the beads up to 50 times their original volume, to a diameter of 1.5-6mm.

Unit dimensions of polystyrene foam beads depend on the dimensions of the vessel where the pre-expanded beads are placed, expansion period and the heat of the steam. With this method, the polystyrene foam beads can be produced in different sizes and densities.

The dimensions of polystyrene foam beads used in this study are between 2 and 4 mm and average 3 mm. Since in previous research, it was found that the dimension of the beads have no effect on the resistance of the concrete produced, the effect of the dimensions were not examined (Lightweight Concrete 1975). It was accepted that the apparent density of the polystyrene foam beads has no effect also on the resistance, because the density of the beads are so lower than the density of the gypsum, that the density of the beads are negligible when the whole composite material is considered. In order that the material should be produced economically, the apparent density of the beads should be about $12\text{--}15\text{ kg/m}^3$ (1). The polystyrene foam beads used in this research are maintained made by a water and thermal insulation firm and their volume weight is $\Delta = 11.17\text{ kg/m}^3$. The density of the same material is 18 kg/m^3 .

3.2 Gypsum

The gypsum used in this research is a commercial gypsum hemi-hydrate ($\text{CaSO}_4 \cdot \frac{1}{2} \text{H}_2\text{O}$) product, which is commonly used for internal decorations. From the preliminary tests on this material, it has been found out that, the material starts to set after 10 minutes and ends setting after 16 minutes, using a normal consistency and 60% water/gypsum ratio. Only distilled water was used in the preparation of the specimens and no further retarders or other additives were added. The compressive strength of this gypsum was 16 MPa (160 kgf/cm^2) after its setting time.

3.3 Bonding additive

Adhesion between the polystyrene foam beads which are organic and inorganic based mineral binders, is very low. To overcome this problem, a bonding agent had to be used. As bonding agents only plastic dispersions or adhesive resins can be considered, since these have no adverse effects on the

gypsum properties and do not attack the polystyrene beads. In this study an epoxy based bonding agent (produced by Köster company ECC Resin), is used. Using this material, delayed the setting time of the gypsum and in addition, increased the mechanical strength and moisture resistance of the gypsum matrix.

4 Preparation of the test samples

The mixing of low-density aggregates and mineral binders, may sometimes cause difficulties. Therefore, it is preferred not to keep the matrix elements very fluid and viscous. In the production of the gypsum composite, sand and similar fillers are also not used. As a result, it is hard to exceed the maximum application of polystyrene foam over 600 kg/m^2 .

In the production of the gypsum composite it is not easy to mix gypsum, water and expanded beads because of the great density differences between these two materials. In order to maintain a homogeneous mixture, the surface of polystyrene foam beads must be covered with the gypsum binder. The bonding additive should ensure that the gypsum binder actually coats the surfaces of the beads and not accumulate in the cavities. For this reason various kinds of polymer dispersions or organic based glues can be used that won't affect the setting of the gypsum. For example, PVAc, acrylic and epoxy dispersions can be used in gypsum without a problem.

In the production of test samples Köster ECC epoxy emulsions are used as the bonding additive. The specimens for the trials are prepared in cylindrical moulds of 10 cm in diameter and 20 cm height. Bonding additive proportion is designed as 1/40 of the mixing water. The densities of the gypsum samples have been designed as 200 kg/m^3 , 300 kg/m^3 , 400 kg/m^3 , 500 kg/m^3 , 600 kg/m^3 , 700 kg/m^3 , and 800 kg/m^3 . The calculated proportions of the matrixes of the materials are shown in Table 1.

Table 1: The matrix of the 1 m^3 gypsum compound and its proportions

Density (kg/m^3)	Gypsum (kg)	Polystyrene foam (kg)	Water (lt)
200	152	15	91.2
300	235	15	141
400	317	15	190
500	400	15	240
600	482	15	290
700	565	15	339
800	647	15	388

In the production phase it was found that these proportions needed changing, as a result of the beads receding from each other. The proportion amounts were decreased. The real matrixes that are taken for 1 m^3 composite samples are shown in Table 2.

Table 2: The real mixing values of 1m³ gypsum composite

Density (kg/m ³)	Gypsum (kg)	Polystyrene beads (kg)	Water (lt)
200	162.000	11.141	97.080
300	249.550	11.141	117.770
400	336.450	10.822	159.150
500	424.300	10.742	200.530
600	511.520	10.663	241.910
700	599.380	10.584	257.830
800	606.590	10.584	318.310

The mixtures prepared in the ratios mentioned above were poured into 10cmx20cm cylindrical moulds. It is found out that the epoxy emulsion which is added into the gypsum mixing water, with the proportion of 2-2.5% decreases the amount of mixing water and increases the flow property and the setting time of the specimens. However the high prices of epoxy resins (approximately 37 DM/kg) forces further new researches on other emulsions although a quite small quantity is required for the gypsum and polystyrene composites. The moulded samples were cured in an oven at 40°C for 24 hours to be dried and then their volume units are measured. The results are summarised in Table 3

Table 3: The average volume weights of the cast test samples

Prepared (kg/m ³)	200	300	400	500	600	700	800
Result (kg/m ³)	208	265	397	493	577	655	690

The heat transfer, water vapour permeance, diffusion resistance factor and mechanical strengths of the test samples are evaluated according to the unit volume mass. The real material proportions in 1 m³ gypsum composite according to the volume weight values are given in Table 4

The specifications of the gypsum composite are evaluated according to the volume ratios of the materials that are present in the composite.

Table 4: The gypsum and polystyrene foam matrixes in 1 m³ gypsum composite

Density of the composite material (kg/m ³)	Polystyrene foam beads (in kg)	Gypsum matrix (in litre)	Polystyrene foam beads (in kg)	Gypsum matrix (in litre)	Cavity limits (in litre)
208	11.100	198.900	616.67	164.00	220.00
285	9.550	315.310	530.55	196.35	273.00
397	10.460	387.041	581.11	297.72	121.17
493	10.272	482.791	570.67	371.38	57.95
577	9.957	566.743	553.17	435.96	10.87
655	9.575	645.360	531.94	467.39	0.67
690	8.120	680.847	451.11	519.73	29.00

5 Properties of polystyrene foam gypsum composite

5.1 Segregation

Polystyrene foam gypsum can be used anywhere light-filling material is needed in construction. Mixtures with a density of 200 kg/m^3 , density is the lowest are not satisfactory in construction and have the tendency to segregate.

Examples over this density do not segregate easily because of the rich amount of gypsum surrounding in their composition. That's why more gypsum should be used in order to increase the density. In the samples with the density of $550\text{--}600 \text{ kg/m}^3$, the amount of the cavities are very little and polystyrene foam beads usage is dense enough so that the material reveals high compressive strength. (Figure 1).

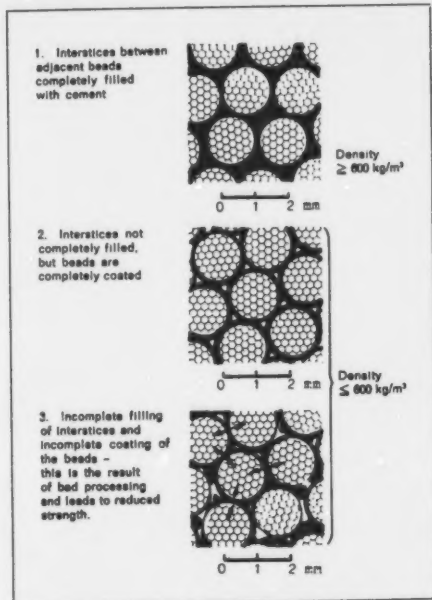


Fig. 1: The structure of gypsum composite (Köhling 1960)

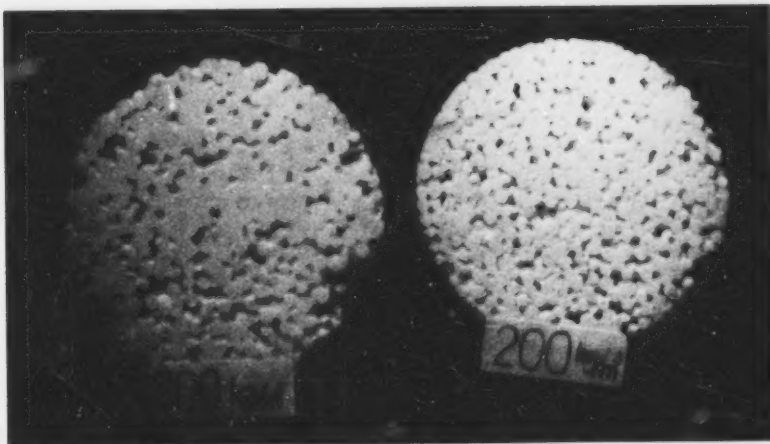


Fig. 2: Gypsum composite samples with different densities

The physical and mechanical properties of the material differ with its density. Naturally, as the density of the material increases, the mechanical strength and vapour diffusion resistance factor increases and the heat transmission ratio decreases.

5.2 Mechanical properties

Since these materials will not be used in flexure or tensile conditions, their values in these aspects are not investigated.

The compressive strengths of the cylindrical test samples with 10 cm diameter and 20 cm height were tested on an MFL Brand 100 kN Universal tester, and the following results were found.

Table 5: Compression strengths related to the densities in average

Density (kg/cm ³)	208	265	397	493	577	655	690
σ MPa	0.183	0.187	0.556	1.34	1.71	2.47	2.74

5.3 Heat transmission values

In the composite polystyrene foam has the largest volume and it is known as the best heat insulating material. Its heat transfer ratio averages 0.040 W/m²°C depending on the moisture content. Gypsum's heat transfer ratio differs with its density and humidity. In preparation of the samples, water/gypsum ratio is taken as 0.47-0.50. The density of the gypsum prepared with this ratio is found as 1.2 g/cm³ and the heat transfer of this material is $\lambda=0.545$ W/m²°C (Hohwiller and Köhling 1968). Gypsum-polystyrene foam bead and air mixture's heat transfer ratio found in composites are calculated according to the average values calculation which are given in Table 6.

Table 6: Heat transfer values related to the densities

Density(kg/cm ³)	208	265	397	493	577	655	690
λ (W/m ² °C)	0.183	0.187	0.556	1.34	1.71	2.47	2.74

5.4 Vapour transfer resistance factor

In polystyrene foam gypsum samples up to 400 kg/cm³ empty spaces that are related with each other are formed. (Figure 1). Therefore their vapour transfer ratios are relatively small. However the vapour transfer resistance of the surplus polystyrene foam beads are very high. In the experiments conducted in the laboratory, the vapour transfer resistance of surplus polystyrene foam are found between 90-150. On the other hand the μ ratio of polystyrene foam panel differs between 25-40 depending on its density. In this study the vapour transfer resistances of the samples having the volume weights of 397kg/m³ and 577kg/m³ are measured. The average value for the samples with a volume weight of 397kg/m³ is $\mu=20$ and for 577 kg/m³ samples $\mu=25$.

5.5 Behaviour against fire

Polystyrene foam beads used in the samples have the tendency of burning easily. But when they are covered with gypsum they do not burn so easy. In samples up to 400 kg/m³, the risk of burning is higher compared with the heavier

samples. In an experiment conducted under laboratory conditions, 6 samples, 40x40x10 cm, were exposed to open fire and has been observed. After half an hour, the polystyrene foam beads on the surface of the blocks which had direct contact with fire, evaporated without burning, and the surface temperature of the samples did not exceed 30 °C. According to these results, gypsum panels of 10 cm thickness and with a density of 500 kg/m³ have the ability to retard fire for half an hour.

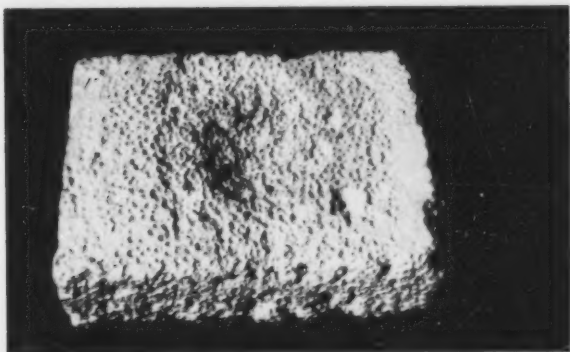


Fig. 3: Behaviour of gypsum-polystyrene foam composite against fire

6 Conclusion

The conclusions of the experimental program which was performed to examine the durability of lightweight gypsum with polystyrene foam beads, can be summarised as follows:

- The lightweight gypsum with polystyrene foam is a perfect heat insulating material with good mechanical strength which can be used in self bearing construction elements, combining thermal insulation with the static function.
- If the lightweight gypsum is designed as a sandwich panel, having an internal layer of lightweight gypsum with polystyrene foam, and gypsum sheets on both faces, it will work as a very good fire delaying element
- Because the material itself is lightweight, there is a considerable reduction of weight in the buildings.
- The Composite of gypsum with polystyrene foam can be used as a filling and heat preserving material in many areas of construction. Among the application areas considered are as a base material in floors that touch the base, in normal floors as elastic and light filling material, load bearing panel structures. It can be used in ceilings as a heat preserving element and in sandwich elements with an internal layer of lightweight gypsum.
- In restoration of historical wooden buildings, using polystyrene foam gypsum will maintain heat insulation as well as increasing the fire

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protection which is a very important factor to wooden buildings. In addition polystyrene foam gypsum blocks can be used as a separation or filling wall material.

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48 BEARING STRENGTH OF CONCRETE CONTAINING POLYSTYRENE AGGREGATE

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Abstract

Concrete has the ability to withstand a higher concentrated load over part of its area than the load placed over its entire area. Previous studies on the bearing capacity of normal weight concrete indicated that the bearing strength is related to the compressive strength and the total area to bearing area ratio. This paper reports the results of an experimental investigation into the bearing strength of polystyrene aggregate concrete as a function of the bearing area and compressive strength.

Expanded polystyrene granulates can be used with the conventional concrete making materials to produce lightweight polystyrene aggregate concrete, having a wide range of performance characteristics. Normal weight high-strength concrete mixture, having the compressive strength of about 80 MPa, was modified to produce reductions in density of concrete by replacing varying amounts of the coarse aggregate with expanded polystyrene beads. Using 200 mm concrete cube specimens, the bearing strength of polystyrene aggregate concrete was determined. Square and circular bearing area shapes were used and the bearing area was varied from one-half to one-eighth of the total area. The bearing strength (f_b) for polystyrene aggregate concrete is given by the following empirical expression: $f_b = (0.635 + p) f_c' (R)^{0.5}$ where p is a coarse aggregate replacement level by polystyrene, R is the ratio of total area to bearing area and f_c' is the concrete compressive strength obtained from a cylinder test. The failure pattern under the bearing load was found to vary with the decrease in the density of polystyrene aggregate concrete. The lightweight polystyrene particles are found to enhance the durability of concrete when it is either subjected sulphate attack or freeze-thaw cycles.

Keywords: Bearing strength, compressive strength, durability, expanded polystyrene, failure pattern, lightweight concrete

1 Introduction

Lightweight expanded polystyrene in the form of spherical beads or irregular granulates can be used with conventional concrete making materials to produce lightweight concrete mixtures with a wide range of densities. The polystyrene particles have a negligible water absorption capacity due to its closed cellular structure. Cook (1973) reported that the use of untreated polystyrene particles had created difficulties in achieving a uniform dispersion in the concrete matrix due to the hydrophobic nature as well as to the lightness of the polystyrene. Sri Ravindrarajah and Tuck (1994) reported that when the polystyrene beads are coated with an inert chemical, having the hydrophilic property, segregation is eliminated and the polystyrene aggregate content can be easily increased to produce lightweight concrete.

Bischoff et. al. (1989) reported that the polystyrene aggregate concrete has an excellent energy absorbing property, hence it is suitable for applications to reduce the effect of impact loading. Sri Ravindrarajah and Naji (1993) reported that the polystyrene aggregate concrete is capable of accommodating sulphate contaminated materials without significant damage. Lai et. al. (1996) studied the behaviour of reinforced concrete beams with polystyrene aggregate concrete and the results showed increase in short-term deflection with no change in the cracking behaviour when compared with similar beams having the normal weight concrete with equal strength.

Concrete has the ability to withstand a higher concentrated load over part of its area than of a load placed over its entire area. The bearing capacity of normal weight concrete has been widely studied by Meyerhof (1953), Shelson (1957), Tung and Baird (1960), Hawkins (1968), Haagsma (1969), Niyogi (1973), Williams (1979) and Lieberum (1989). A cone or failure wedge is formed under the loaded area and this moves downward and causing splitting of the loaded specimens. The purpose of this study is to investigate the effects of the inclusion of polystyrene beads on the bearing capacity and failure process of concrete.

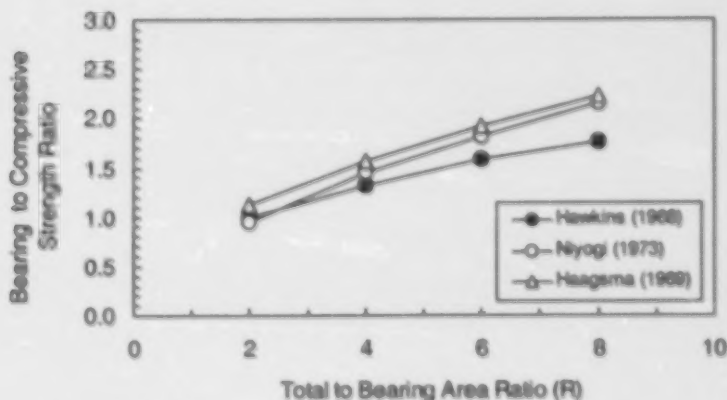


Fig. 1: Bearing strength of normal weight concrete as a function of compressive strength and bearing area

2 Background

It is known that the concrete is able to higher bearing stresses than its compressive strength. Tung and Baird (1960) based on Coulomb/Mohr theory of rupture suggested the failure under bearing stresses as a sliding process along planes inclined to the direction of the principal stresses. The resistance to sliding is brought about by the strength of internal cohesion of the material and by the resistance of each constituent to internal friction. Lieberum and Reinhardt (1989) reported that allowable stresses under concentrated loading conditions could be higher than the predicted design stress due to the improvement in concrete strength when the transverse strain is confined. Figure 1 shows the relationship between ratio of bearing strength to cylinder strength and cylinder strength proposed by Hawkins (1968), Niyogi (1973) and Haagsma (1969).

The proposed equation for bearing strength of concrete by Haagsma (1969) is similar to that adopted by ACI Building Code (1989). The bearing strength (f_b) $\approx 0.79 f_c (A_c/A_b)$, where f_c is the compressive cube strength, A_c is the total area of concrete and A_b is the area of concrete under the bearing load. An increase in the ratio of the total area to the bearing area produces a significant increase in the bearing strength due to the confinement provided by the non-loaded area of concrete. Tests conducted by Meyerhof (1953) displayed sliding failure in concrete due to the formation of a distinctly separate cone under a circular base plate. Shelson (1957) observed the formation of an inverted pyramid in concrete under small base plates and under larger base plates an internal pyramid was formed.

3 Experimental details

3.1 Materials and mixture compositions

General purpose Portland cement (Type GP), similar to ASTM Type I, conforming to AS 1319 was used as the binder material in all concrete mixtures. Crushed basalt (specific gravity of 2.65), having a maximum aggregate size of 10 mm, and coarse river sand (specific gravity of 2.62) were used as coarse and fine aggregates, respectively. A superplasticiser (high-range water-reducing admixture) consists of a salt of naphthalene sulphonate formaldehyde condensate was used. Spherical shaped expanded polystyrene beads (specific gravity of 0.67) with a mean diameter of 3 mm was used. They were coated with an inert chemical to improve the adhesion and dispersion of them within the concrete matrix.

The control concrete mixture is a normal weight high-strength concrete. It had the following mixture compositions: 460 kg/m³ of cement; 161 kg/m³ of water; 735 kg/m³ of river sand; and 1105 kg/m³ of basalt. The water to cement ratio of this mixture is 0.35 and 270 ml/m³ of the superplasticiser was used to achieve sufficient workability. By replacing 10, 20 and 30% of the coarse aggregate by solid volume with the polystyrene beads, three polystyrene aggregate concrete mixtures were produced. The quantities of the superplasticiser used were 280, 300 and 340 ml/m³ for the concrete mixtures with 10, 20 and 30% of coarse aggregate replacements with the polystyrene aggregate, respectively.

3.2 Batching, mixing and preparation of test specimens

Experience had shown that a proper mixing sequence is necessary to produce a uniform polystyrene aggregate concrete mixture. Coarse aggregate, fine aggregate and cement were first placed in a pan-type mixer and dry mixed for about five minutes. Then, the required amount of the polystyrene beads was added and the dry mixing was continued for another two minutes. Finally, the water was gradually added to the mixture while the mixing process was in progress. The superplasticiser was introduced by thoroughly mixing it with the mixing water.

For each concrete mixture, a number of 100 mm diameter by 200 mm high standard cylinders were moulded in steel moulds for compressive strength testing at the age of 28 days. In addition, a number of 200 mm cubes were moulded in specially-made timber moulds for the bearing strength test. A table vibrator was used to compact the fresh concrete in the moulds.

3.3 Curing and testing of specimens

The test specimens were stripped from their moulds after 24 hours and stored in a water bath at $23 \pm 2^\circ\text{C}$ until the age of testing at 28 days. The cylinders were tested in direct compression while the cubes were subjected to bearing load through either circular or square 15 mm thick steel plates. The bearing area was varied to produce the ratios of total area to bearing area of 2, 4, 6 and 8. The bearing plates used are 70.7, 81.7, 100 and 141.4 mm square and 79.8, 92.2, 112.8, and 159.6 mm diameter circular in shape. The bearing plates were positioned centrally on one of the formed surfaces of the 200 mm cubes and the compressive load was gradually applied on the cubes at a rate of 60 MPa per minute through the bearing surface. For each concrete mix, two identical specimens were tested for the same total area to bearing area ratio (R) and the same shaped base plate.

4 Results and discussion

4.1 Workability and unit weight of polystyrene aggregate concrete

Although no workability test was conducted on fresh concrete mixtures, visual examination and handling experience indicated that the polystyrene aggregate concrete mixtures became stiff and rubbery with the increase in the polystyrene aggregate content. In addition to the use of vibrating table, it was found necessary to use a modified tamping rod with a circular disc attached to achieve a proper compaction. While finishing the moulded specimens, some difficulties were encountered to obtain smooth surface. The polystyrene aggregate particles caused small lumps on the surface of the concrete and this proved difficult to trowel.

The control concrete mixture had the unit weight of 2455 kg/m^3 . When the coarse aggregate particles were replaced with polystyrene beads by equal *solid* volumes corresponding to 10, 20 and 30%, the unit weight (wet density) of the concrete mixture was reduced to 2330, 2210 and 2080 kg/m^3 , respectively.

It was noted that the measured values for the unit weight were lower than those expected due to entrapped air voids in compacted concrete. The entrapped air content for the control mix was 0.41% and for the polystyrene aggregate concrete mixtures were 1.06, 1.56 and 2.80%, respectively for the mixtures having the aggregate replacement levels of 10, 20 and 30%, respectively.

4.2 Compressive strength of polystyrene aggregate concrete

Unit weight is the most important physical property of concrete in relation to the strength of concrete. Figure 2 shows the effect of unit weight on the cylinder strength of concrete. As expected, a decrease in the unit weight of concrete produced a significant reduction in the cylinder strength of concrete. When the unit weight decreased from 2455 to 2330 kg/m^3 (a decrease of 5%), the cylinder strength dropped from 79 to 56 MPa, (a drop of 29%). When the unit weight decreased by 10 and 15%, the cylinder strength decreased by 51 and 65%, respectively. This showed that the relationship between the void content and compressive strength is non-linear. The 28-day cylinder strength for the polystyrene aggregate concrete with the lowest unit weight of 2080 kg/m^3 is 28 MPa, hence this concrete can be considered as a lightweight structural concrete.

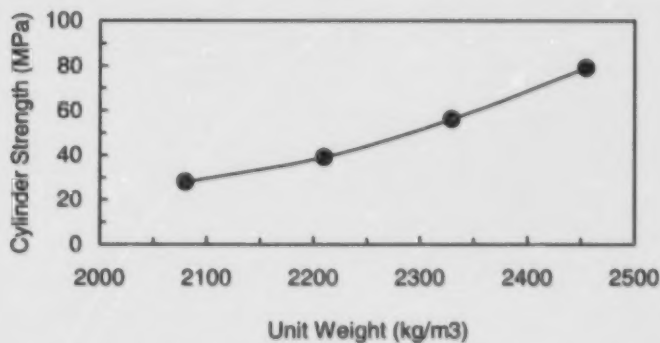


Fig. 2: Relationship between unit weight and compressive strength

Table 1: Bearing strength (MPa) of polystyrene aggregate concrete

Polystyrene Content (%)	Unit Weight (kg/m^3)	Cylinder Strength (MPa)	Bearing Area Shape	Bearing Strength (MPa)			
				R* = 2	R = 4	R = 6	R = 8
0	2455	79	Square	80	96	120	136
			Circle	80	105	119	138
10	2330	56	Square	62	74	101	114
			Circle	59	80	96	111
20	2210	39	Square	51	66	82	96
			Circle	52	65	82	95
30	2080	28	Square	39	53	66	75
			Circle	40	52	67	74

*Note: R = Total Area / Bearing Area

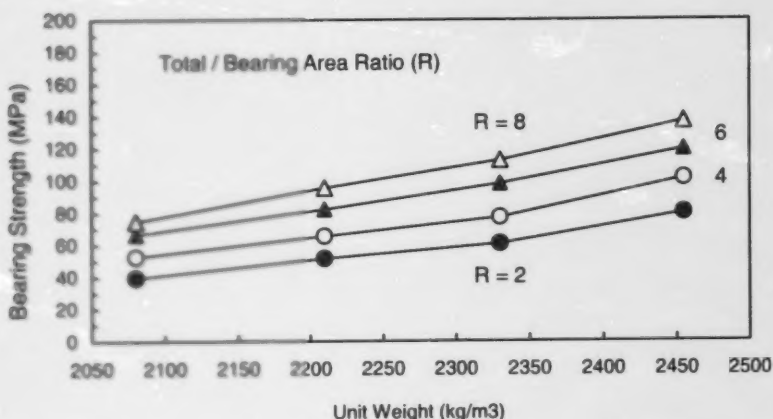


Fig. 3: Effect of unit weight, bearing area on bearing strength of polystyrene aggregate concrete

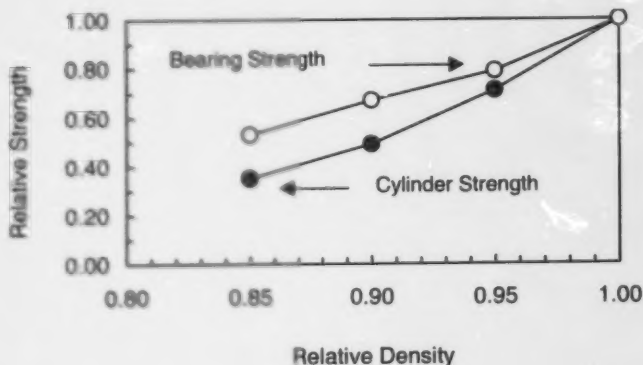


Fig. 4: Relationship between relative strengths in compression and bearing and relative density for polystyrene aggregate concrete

4.3 Bearing strength of polystyrene aggregate concrete

Table 1 summarises the results obtained from bearing strength tests for the control concrete and three polystyrene aggregate concrete mixtures when square and circular shaped bearing plates were used. The total to the bearing area ratio was varied from 2 to 8. For a given total to bearing area ratio (R), the bearing strength values obtained with the square and circular base plates do not differ significantly. Figure 3 is plotted using the mean values obtained with both square and circular base plates. Hence, each point in this plot is the mean of four results obtained with four identical test specimens. The results shown in Fig. 3 indicate that the bearing strength increased with the increase in the total to the bearing area ratio, independent of the unit weight of concrete.

Figure 4 shows the bearing and cylinder strengths of polystyrene aggregate concrete in relation to those for the control concrete. For the bearing strength, each point shown in this plot is the mean of 16 readings taken with both square and circular base plates having different sizes, whereas the compressive strength is the mean of 3 readings from three identical cylinders. Similar to the compressive strength, a decrease in the unit weight produced significant reductions in the bearing strength of concrete. However, the compressive strength is more sensitive to the unit weight than the bearing strength. When the unit weight is decreased by 15%, the compressive strength and the bearing strength decreased by 65% and 47%, respectively.

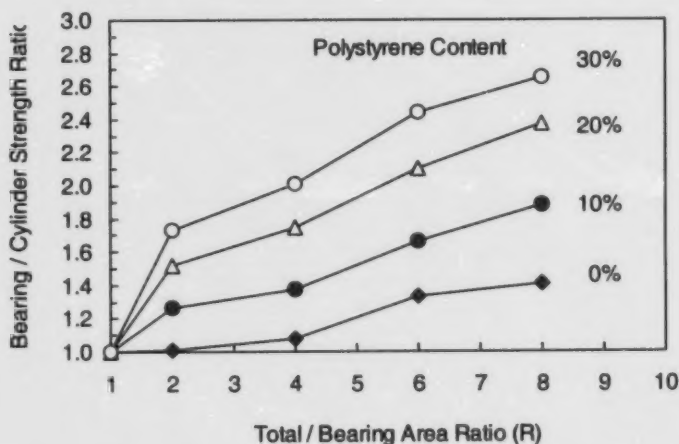


Fig. 5: Relationship between bearing to cylinder strength ratio and total to bearing area ratio (R)

4.4 Relationship between bearing strength and cylinder strength

Figure 5 shows the relationship between the ratio of bearing strength to cylinder strength and the ratio of total area to bearing area for the control concrete and polystyrene aggregate concretes. For a given ratio of total area to bearing area, the polystyrene aggregate concrete showed significantly higher values for the ratio of the bearing strength to the cylinder strength than those for the control concrete. With the increase in the ratio of total area to bearing area, the polystyrene aggregate concrete showed significant bearing strength in relation to its compressive strength. For the polystyrene aggregate concrete with the unit weight of 2080 kg/m³ and the bearing area of one-eighth of the total area, the bearing strength is 2.66 times of cylinder strength. However, for the control concrete for the same bearing condition the corresponding value is only 1.48.

As mentioned earlier, ACI Building Code (1989) considered that the ratio of the bearing strength to the compressive strength is proportional to the square root of the ratio of the total area to the bearing area. Using the results obtained in this investigation, the following empirical equation was developed for the bearing strength (f_b) of polystyrene aggregate concrete in relation to the cylinder strength (f'_c), polystyrene aggregate content and the total area to bearing area ratio.

$$f_b = (0.635 + p) (A_c/A_b)^{0.5} f_c' \quad (1)$$

The factor 0.635 represents the governing design value that incorporates the mixture parameters and testing arrangements. Haagsma (1969) proposed a factor of 0.79 when cube strength is used. Since the cylinder strength is 80 per cent of cube strength, then the factor changes to 0.632, when cylinder strength is considered. The second factor 'p' is equal to the polystyrene aggregate content in relation to coarse aggregate replacement. As an example, when 20% of coarse aggregate is replaced with the polystyrene aggregate, the value for 'p' is 0.20. It was found that as the ratio of the total area to the bearing area is increased, the predicted bearing strength using Eqn. (1) is within ± 5 MPa of the measured values.

4.5 Bearing strength design consideration

AS3600 (1994) states that the design bearing stress at the concrete surface should not exceed $[\phi 0.85 f_c (A_c/A_b)^{0.5}]$ or $2 \phi f_c$ (which ever is less) where A_c is the bearing area and A_b is the largest area of the supporting surface that is geometrically similar to and concentric with A_c . ϕ is the strength reduction factor of 0.6, which is lower than 0.7 used by ACI Code (1989). Comparing the results obtained in this study, it can be shown that both ACI Code (1989) and AS3600 (1994) are applicable to the polystyrene aggregate concrete as well.

4.6 Failure process under bearing loading

Under the application of the bearing load, it was noticed that shortly before the failure occurred vertical cracks initiated at the top of the test cubes and propagated downward in each of the four sides or corners indicating splitting due to sliding failure. The maximum load was reached shortly after the cracks appeared and started to propagate. Typical cone failure of the specimens was noted for both square and circular base plates. The failure wedge under the base plate was of conical shape for the circular plate whereas a pyramidal shape was observed for the square plate.

When the bearing area is half that of total area, cone type failure was not observed. The failure appeared to be a sliding failure in a vertical direction. This may have occurred due to the limited amount of transverse confinement available.

The failure for the control concrete under bearing load was sudden, indicating the brittle nature of high-strength concrete. With the inclusion of polystyrene aggregate the failure was more gradual, indicating the energy absorbing capability of the expanded polystyrene aggregate particles. For the polystyrene aggregate concrete, the base plates were found to move into the specimens considerably under the increasing bearing load. Once the maximum load was reached, the load carrying capacity was gradually decreased due to the ductile behaviour of the polystyrene aggregate concrete. After the failure, the cracked sections remained intact.

The circular and square plates produced different initial crack initiation and propagation patterns on the specimens under the bearing load. Under the circular plates cracks tend to propagate to the nearest edge of the specimens from the base plate, whereas with the square plates the cracks propagated towards the extreme corners or near corners of the specimens. These cracks are formed the results of the stress concentrations developed at the corners of the square plates.

4.7 Durability of polystyrene aggregate concrete

The studies reported by Sri Ravindrarajah and Naji (1993) indicated that polystyrene aggregate concrete is capable of allowing higher proportions of sulphate ions without causing significant expansion than that allowed for the control normal weight concrete. Sabaa (1998) reported that the polystyrene aggregate concrete with the density between 1750 and 1850 kg/m³ showed no reduction in its dynamic modulus of elasticity when it was subjected to 450 cycles of freezing and thawing. Hence, these studies indicate that polystyrene aggregate concrete is more durable under severe exposure conditions than the normal weight concrete. The polystyrene aggregate particles act as energy absorbing component in the concrete system, thus negating the disruptive expansive stresses created by the changes in the exposure conditions or by the chemical attack.

5 Conclusions

- a) The bearing strength of polystyrene aggregate concrete is increased with an increase in the ratio of total area to bearing area. With the increase in the polystyrene aggregate content, the ratio of the bearing strength to the compressive strength increased
- b) An increase in the polystyrene aggregate content reduced the bearing strength of concrete but to a reduced extent when compared with the compressive strength.
- c) The bearing strength of polystyrene aggregate concrete can be predicted by using the equation: $f_b = (0.635 + p) (A_c/A_b)^{0.5} f_c'$, where f_c' is the cylinder strength, 'p' is the polystyrene content based on coarse aggregate replacement, A_c and A_b are the total area and bearing area, respectively.
- d) ACI 318 (1989) and AS3600 (1994) are conservative in their approach to design requirements for bearing strength of concrete and they are equally applicable to the polystyrene aggregate concrete.
- e) The base plate shapes used, square and circular, had no significant influence on the bearing strength for the polystyrene aggregate concrete, although different failure patterns were observed.
- f) Inclusion of expanded polystyrene aggregate particles induces ductility to the concrete and the failure process was changed from brittle to ductile due to the energy absorption capability of the polystyrene particles.
- g) Polystyrene aggregate concrete is more durable when it is subjected to sulphate attack or freeze-thaw cycles.

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49 COMPOSITE SCRIM REINFORCED-CEMENTITIOUS BOARDS: ACCELERATED AGING AND PERFORMANCE PREDICTION

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Abstract

Composite cement board emerged recently as one of the more innovative and highly promising engineered building materials. These cementitious boards usually consist of an aggregated Portland cement-based core matrix, reinforced with glass fiber scrims embedded on both sides of the board. To prevent alkali attack on the glass in the high pH environment of the cementitious matrix, the scrim is coated using specially formulated PVC plastisols. This paper reports on recent testing and a comparative assessment of non-woven and woven plastisol-coated glass fiber fabrics. Long term performance and durability of cementitious boards reinforced with such scrims were studied. An adaptation of BRE SIC method was used to simulate long term exposure conditions that construction materials can be exposed to in the field. The aging process was followed by an evaluation of the structural properties of the conditioned board specimens as well as by scanning electronic microscopy (SEM) examination. It is shown that the newly developed non-woven glass-fiber scrim reinforcements have excellent long term durability characteristics, fully comparable to the woven scrims. Superior board strength retention and aged glass fabric fiber surfaces, essentially free of signs of any extensive alkali attack, demonstrate a definite potential and suitability of glass laid scrims for reinforcement of cementitious boards in many engineering and construction applications.

Key words: Cement board; glass fiber scrim reinforcement; functional requirements, durability; simulated aging; non-woven scrim; woven scrim.

1 Introduction

Composite cement boards were developed and introduced to the building materials markets during the late 1970s and early 1980s. The impetus for their development was the unsatisfactory long term durability (LTD) performance of gypsum- or wood-based building boards when exposed to elements, water or high humidity. They deteriorate rapidly when exposed to continuous wetting. The cementitious boards were initially introduced for interior applications potentially subjected to high moisture or water exposure, such as tile backerboard (Reicherts and Venta, 1984), or high abuse areas, such as corridor walls, but they are increasingly utilized in exterior applications, such as exterior insulation and finish systems (EIFS), as well. It is estimated that the production capacity in the U.S.A. and Canada is approaching 75 million m²/year, with the actual production growing 20% annually. Currently, there are five major producers of cement boards in North America, and similar products are starting to make some inroads overseas as well.

Although different proprietary production processes are used, all these cementitious boards consist of aggregated Portland cement-based core matrices, usually in the 1200–1280 kg/m³ density range, reinforced with glass fibre scrims. The nominal thickness of the boards is usually 12.7 mm. All of these cement boards employ two layers of PVC-coated glass fibre scrim embedded on both sides of the board (face and back), just under the surface. The use of glass fibre-based scrim - as opposed to glass fibres uniformly distributed throughout the cementitious matrix - was selected both to maximize the reinforcement effect on the flexural properties of the boards and to facilitate encapsulation of the glass in the polymer coating.

2 Woven vs. non-woven reinforcements

In the work reported here, we studied the relative performance of two types of fibreglass yarn-based reinforcements, non-woven versus woven scrims. Since the inception of cementitious boards the reinforcing scrim, sometimes also called mesh or fabric, has usually been a woven product using E-type glass fibre-based yarn. To prevent alkali attack on the glass in the high pH environment of the cementitious matrix, the yarn is coated during manufacture using a specially formulated PVC plastisols in a two-step process. Single-end, PVC-coated yarns are only then woven into the scrim of desired configuration, tendered and fused together. The amount of PVC plastisol used to protect the glass fibre fabric is substantial; weight ratios of plastisol to glass typically range from 60/40 to 70/30. Clearly, this multiple step process is a rather expensive technology in terms of material and manufacturing costs, and as such, the reinforcing scrim can represent 50% to 75% of the total raw material costs of cement board production.

In response to these issues, as an alternative, non-woven glass meshes were developed. Such materials are first "assembled" (again using ordinary E-glass), then dipped in a plastisol that serves as both the glue that binds the glass fibre yarns together, and as a protective coating against alkali attack. As the manufacturing

process for non-woven materials is significantly simpler, there is a substantial economic advantage to using such a product in cement boards.

Development of these non-woven, so-called "laid" scrims, has posed a significant challenge. The initial experience on the board manufacturing line has been poor, largely because the process introduces stresses from tensioning of the fabric, which lead to the development of small cracks in the coating, especially at the crossover of the glass yarns running in the x and y directions. These small cracks in the coating sheath allow penetration of alkali and its attack of the glass fibres, leading to sub-standard long-term durability and ultimately, failure of the cement board. Other factors affecting the efficiency of the polymer coating as a barrier against alkali attack, affecting both woven and non-woven scrims, are the chemical nature and quality of the plastisol coating itself, and the process for its application to the glass yarn. Even small pinholes in the coating will affect its performance. Furthermore, in the non-woven dipped mesh, it is difficult to properly coat the area where two yarns intersect. The interior surfaces in this region are often starved for binder/coating, allowing deterioration in the aggressive alkali environment.

It has been only over the last few years that through major cooperative R&D efforts between the laid scrim producers such as Bayex Division of Bay Mills Limited and leading cement board manufacturers, non-woven fabrics have been successfully introduced into cement boards. While in the early days of the cement board development non-woven scrims were considered inferior to woven reinforcements, results of the study reported here confirm the fact that the modern non-woven scrims are equal in their long term durability performance to the woven.

3 Long term durability testing methodology and materials

3.1 Methodology

To assess and compare the long-term durability (LTD) of cementitious boards reinforced with different glass fibre-based materials, test coupons of such boards are subjected to accelerated aging conditions. At different stages of the accelerated aging test, specimens are removed and subjected to testing in flexure. We used so-called "hot water" conditioning and testing methodology, adapted from procedures reported earlier (Janninck, Venta, 1985; Venta, Hemmings, Cornelius, 1995), and loosely based on SIC ("strand-in-concrete") testing methodology established during the development of AR glass fibres (Oakley, Litherland, Proctor, 1981; Litherland, 1985). This method, as described in detail elsewhere (Venta, Cornelius, Ling, Porter, 1997), is currently being balloted by the ASTM Committee C27 for an inclusion into the ASTM standards.

Test samples, with an equal weight of deionized water, are sealed in a plastic bag and placed into a temperature-controlled water bath at 80°C. [There are strong indications that 1 to 2 days of conditioning at 80°C leads to similar deterioration as one year at room temperature water immersion, and 4 to 5 years of natural weathering (Litherland, 1985). At various time intervals the samples are removed from the bath, cooled to room temperature and, while still sealed in the plastic bag, tested in flexure.

Flexural strength testing is conducted on the reference point "as is" samples conditioned at 23°C and 50% relative humidity, and on all the specimens subjected to 3, 7, 14, 21, 28 and 42 days of accelerated aging. All the specimens are tested on a computerized stress-strain universal strength testing apparatus. The design of the three-point-loading specimen fixture is based on that described in the ASTM C947 method (ASTM, 1989) and recommended by the PCI Committee on Glass Fibre Reinforced Concrete Panels (PCI, 1987). Both modulus of rupture (MOR) and deflection at break (δ) are calculated and plotted versus time.

3.2 Materials tested

Three different scrims were used in preparation of laboratory-cast cement boards, one non-woven 8x8 laid scrim ("A"), and two woven leno scrims, 9x9 and 16x10 weave construction, respectively ("B" and "C"). All three versions of the scrim tested are currently used by cement board producers for reinforcement of commercial boards. The cementitious matrix was the same for all boards, closely approximating the actual matrix formulation used in the factory produced boards. Its density was in the 1150 to 1200 kg/m³ range. All the boards were cured at 21°C and 50% relative humidity for 14 days prior to conditioning in hot water and testing.

4 Comparative results

Results of comparative testing of laboratory cast boards reinforced alternately with non-woven scrim "A" and woven glass scrims "B" and "C" are given below. All three reinforcements have shown a similar trend as far as their aging pattern is concerned, i.e. all three of them experienced a fairly rapid drop in strength within the first three days of their exposure in hot water, dropping to about 60% of the original strength. Between 3 days and 42 days of continuing conditioning, further changes in the flexural strength of all three types of boards progressed at a much slower pace.

Table 1: Modulus of rupture vs. conditioning time at 80°C

Conditioning Time [days]	Non-Woven Scrim "A" MOR		Woven Scrim "B" MOR		Woven Scrim "C" MOR	
	average	std. dev.	average	std. dev.	average	std. dev.
	[kPa]		[kPa]		[kPa]	
0	9901	1273	7184	343	10983	581
3	5557	383	4357	203	7488	473
7	5474	294	4413	212	6143	503
14	5592	362	4744	210	5902	389
21	5433	363	4206	98	4895	320
28	5399	268	4309	372	4854	420
42	4482	228	4564	288	4426	310

All boards - disregarding the type and the initial strength of the glass scrim reinforcement - following the 42 days conditioning, exhibit remarkably similar strength; MOR of all of them being in the 4500 kPa range. The strength of the "laid" non-woven scrim kept pace with strength of those boards reinforced with the woven materials. As a matter of fact, rather surprisingly, for up to 28 days of conditioning the non-woven scrim outperformed its woven counterparts.

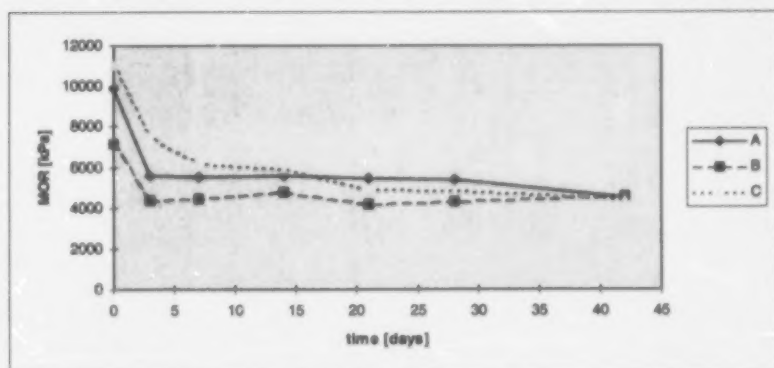


Fig. 1: Modulus of rupture vs. conditioning time at 80°C

Deflection of the cement board at the point of failure is an important parameter for determining its performance in the field. Low deflection of the board indicates board embrittlement and reinforcement deterioration, and potential failure during the handling and application of the board. It would appear that the non-woven scrim provides the cement board not only with better initial peak deflection, but that it retains this advantage throughout the accelerated aging. At the final point, following the 42-days of conditioning, deflection of the "A" scrim-reinforced board is about 34% to 41% higher than that of the two woven scrim-faced boards.

Table 2: Deflection at break vs. conditioning time at 80°C

Conditioning Time Time [days]	Non-Woven Scrim "A" deflection		Woven Scrim "B" deflection		Woven Scrim "C" deflection	
	average	std. dev.	average	std. dev.	average	std. dev.
	[mm]		[mm]		[mm]	
0	16.48	3.35	12.22	1.50	14.78	1.68
3	8.71	1.68	8.56	1.09	9.40	0.79
7	8.31	2.11	6.88	1.12	7.80	0.56
14	7.70	0.53	8.20	0.69	6.91	0.97
21	7.82	0.61	4.27	0.69	5.56	0.97
28	6.38	0.38	3.58	0.48	4.62	0.36
42	4.78	0.94	3.38	0.81	3.58	1.07

Our experience shows that for the cement board to perform satisfactorily in the field, it should have peak deflection of 7.5 mm or higher. It is interesting to note the non-woven "A" scrim provides this much or better deflection for up to 21 days of accelerated aging, while the two woven scrims for only 14 days with the "B" scrim, and in the case of the "C" scrim, rather surprisingly, for only 7 days.

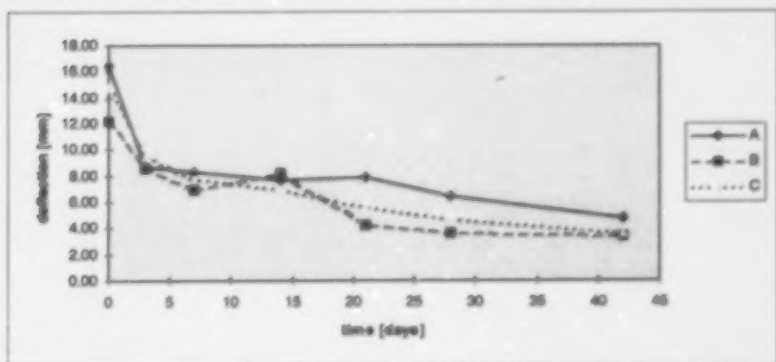


Fig. 2: Deflection at break vs. conditioning time at 80°C

5 Discussion and concluding remarks

The new generation of non-woven "laid" glass fibre scrims contains sufficient coating material and application quality to provide the necessary long-term protection to the glass fibre against alkali attack when used in cementitious boards. Results of our accelerated aging study, with glass fibre-based scrims used as reinforcement of laboratory-cast cement boards subjected to conditioning in hot water at 80°C and tested in flexure, show that the non-woven scrim developed by Bayex is functionally equivalent to older woven reinforcements. In some aspects, specifically in the deflection at break, non-woven scrims are perhaps superior to their woven counterparts.

Thus the cement board producers now have a practical, high performance alternative to the single-end coated woven scrims, and following extensive testing of their LTD, as of today, all the major board producers employ these non-woven fabrics as reinforcement of their products. Considering the fact that the manufacturing of the "laid" non-woven scrims is more cost efficient than the production of the woven fabrics, and that the reinforcement represents a substantial portion of the total raw materials costs, use of non-woven scrims offers a substantial advantage to the board producers.

The continuing improvement of the non-woven scrims has allowed their producers to capture an increased share of the cement board market and it is expected to lead them eventually into the position of preferred suppliers not only to the

established industry in North America, but also to the rapidly developing overseas industry.

6 Future developments

The cement board industry in North America has experienced substantial growth in use of its products for about 15 years now. Cement boards became well established and widely accepted building panels. Fibreglass scrims have been used since the inception of this industry, and proved their worth in providing excellent long term reinforcements to the cement boards. PVC plastisols have allowed sufficient protection to the glass yarns preventing their deterioration in the high pH environment of the cementitious matrices. Nevertheless, there has been a continuous search for other, alternative reinforcements of the cement boards, that would provide either a performance or an economic advantage.

One possible alternative could be reinforcing fabrics made from high zirconia content alkali resistant (AR) fibreglass yarns. Although such glass fibers have been available for years, their high cost and the lack of availability of suitable yarn tex (sizes) did not allow their use in cement boards until now. This could change in the future.

Other alternative fibres have been studied in cementitious applications for some time. Consideration has been given to a wide range of natural and synthetic fibres, such as:

- natural - cotton, hemp, flax, sisal, wood pulp
- synthetic - polypropylene, nylon, polyester, polyethylene, rayon, carbon
- metallic - ferrous, non ferrous

All the above fibres are available in spun staple form. The synthetic and metallic fibres are available in continuous filament form as well. A number of requirements must be considered in the selection process. Principal design factors include:

1. Young's modulus and general stress/strain properties
2. Short term heat resistance
3. Long term alkali and water resistance
4. Economics
5. Processability

Although fibreglass has the inherent, but addressable, shortcoming of poor alkali resistance, alternative fibres also suffer from one or more design factor limitations. For example polyolefins such as polyethylene and polypropylene typically are deficient in the first factor (low Young's modulus). Most metallic fibres are difficult to process and may require some corrosion protection. Many cellulosic fibres do not have the right combination of heat and alkali resistance.

Carbon fibres typically have low breaking strain and economic or processing constraints.

Recently we have reported on some progress in the area of natural and synthetic fibre based reinforcements (Venta, Ling, Porter, 1998). These reinforcements contain fibres which are more inherently resistant to alkali and therefore require less protective coating. Compared to fibreglass reinforcements, these alternatives have less than half the coating levels. Recent LTD results involving proprietary fibre combinations are shown in Table 3. It should be noted that these are preliminary results. These reinforcements are not yet available commercially.

Table 3: Modulus of rupture [kPa]- developmental reinforcements

	"as is"	14 days	28 days
Fibre "X"	5680	6730	6180
Fibre "Y"	5390	5780	6730

The area of natural and synthetic (organic) reinforcement development is very active. The expectations of high performance and durability are increasing. Progress in this area is continuing and we expect to commercialize alternatives to fibreglass reinforcements in the next two years.

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PART TWO

**DURABILITY OF BRICK
MASONRY, STONE AND TILE**



50 PHYSICAL AND MATHEMATICAL MODELLING OF MASONRY DETERIORATION DUE TO SALT CRYSTALLIZATION

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Abstract

Salt crystallization tests have been carried out on brick masonry specimens physically modelling the behaviour of a real brickwall exposed to salt decay due to capillary rise. The reliability of the test has been shown by comparing the decay obtained in laboratory and on site. The surface damage has been measured by a non destructive technique based on the use of a laser sensor as a profile recording device. The loss of material at every measurement has been assumed as a parameter representing the continuous damage. The data collected outline the evolution of the deterioration, over space and time, of the mortar-bricks composite. On this basis a numerical model describing the damage evolution is being studied.

Key words: masonry, salt crystallization, physical and mathematical modelling, stochastic processes, damage.

1 Introduction

Salt crystallization has caused decay of brick and stone masonry at all latitudes. Physical and numerical modelling of the phenomenon can be useful to understand its development and to foresee the extent of the decay so that preventive measures can be taken. Modelling can also help in the case of protective treatments to study their durability and make the right choice of products and treatment techniques.

The aim of the research carried out by the authors is; (i) to realise in laboratory a complete procedure able to produce a damage mechanism similar to the one occurring in a natural environment, (ii) to monitor the damage in a non destructive way and (iii) to implement a probabilistic model interpreting the

Durability of Building Materials and Components 8 (Volume One). Edited by M.A. Lacasse and D.J. Vanier.
Published in 1999 by NRC Research Press, Ottawa ON K1A 0R6, Canada. ISBN 0-660-17737-4

phenomenon. Results have been produced concerning single component materials (stones and bricks), (Garavaglia et al., 1996). Nevertheless it has been shown that the mutual influence mortar-brick/stone in a real wall can highly influence the behaviour of the single components (Binda et al, 1985), (IRPA-KIK et al, 1996). Therefore with an experimental campaign on masonry prisms and applying a non-destructive measurement procedure (Berra et al., 1993) the authors have prepared the basis for the application of the probabilistic model.

Aging tests based on crystallization cycles have been carried out in laboratory using wallettes physically modelling a real brick or stone-masonry wall. Comparisons between the decay obtained in laboratory and on site on a wall surface have been made showing the similarity of the two situations. A non destructive measurement procedure of surface damage with time has been applied using a laser sensor as a profile recording device. The data collected outlines the evolution of the deterioration of the mortar-bricks composite, over space and time. The experimental results have shown some interesting issues: a) the independence between the time to first damage and its magnitude; b) the apparent dependence of the damage and/or its magnitude on the initial roughness of the wallette surface; c) the distribution of the time t_0 to first damage seems to follow a Weibull function; d) the distribution of the first damage seems to follow a Log-normal function; e) the damage process over time $t > t_0$ seems to be a stochastic process. On the basis of these observations a simple time-dependent deterioration model is being implemented.

2 Test description

Within the frame work of an EC Contract (IRPA-KIK et al., 1986) a salt crystallization test developed on TNO in the Netherlands and later recommended by RILEM (RILEM 127MS TC, 1998), was carried out on small wallettes which had been prepared with 7 types of bricks and 5 types of mortar. All together 54 wallettes were tested. For each type of brick and mortar, 6 specimens were prepared and numbered from 1 to 6 with the symbols TNO, KIK, PDM, of the 3 partners. Four of the six specimens were each treated with a different water-repellent, the fifth with a consolidant and the sixth one was kept as a reference without any treatment.

The main steps of the test are described below:

- wallettes are built with at least two mortar courses and placed, in glass or plexiglass box, which is filled up to 20mm from the bottom with a sodium sulphate solution with a concentration of 10% (m/m) anhydrous salt dissolved in pure water.
- After 24 hrs the wallette is taken out of the container which is emptied, cleaned and partly filled with clean and dry gravel (grain size 2 to 4mm); the wallette is replaced on top of the gravel. The spaces around the wallette are closed by hard polystyrene foam strips so that only the upper face of the specimen is exposed to the environment (Fig. 1a,b).

The container is placed in a climatic chamber at 20°C and 50% R.H. and weighed until constant mass. Once the constant mass is reached the test is continued for one month without adding water, then some water is added if a test continuation is needed.



Fig. 1a - Walette KIK 2.6



Fig. 1b - Walette TNO 2.6

3 Measurement procedure

A non destructive procedure was used to monitor the decay of the wallette surface, based on a laser sensor used as a profile recorder. This procedure has already been proposed by the authors together with ENEL-CRIS (Berra et al, 1993). Before starting the test a first profile of the upper surface of the wallette is defined along chosen lines (Fig. 2) is read and drawn with the use of the laser sensor (Fig. 3). Then the weighing procedure up to constant weight is applied; when constant weight is reached a second measurement of the profiles is carried out after brushing off the efflorescences. Fig. 4a shows the variation of one of the profiles during the development of the test. The damage of the surface is measured by calculating the difference between the vertical coordinates of two or more subsequent profiles. The loss is measured as the area included between two consecutive profiles. Bulging of the wallette surface caused by cryptoefflorescence means the beginning of deterioration even if delamination has not yet occurred.

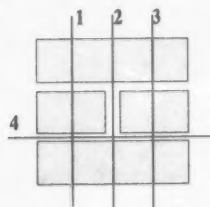


Fig. 2 - Profiles position

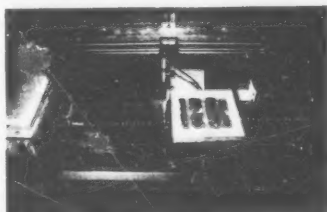


Fig. 3 - Laser sensor

4 Interpretation of the experimental data

Among the wallettes tested only the untreated ones (see Table 1) are elaborated here. In Figure 4a the presence of swelling phenomena along the different profiles is evident. It is a common approach to consider bulging of the surface as a damage: when bulging is measured a separation of a layer of material is started due to the cryptoefflorescence formed beneath the surface.

Table 1: Mortar/brick combinations

Brick	Mortar	untreated
B1	Hydraulic	KIK1.6
B2	Hydraulic	KIK2.6
B1	Hydrated	KIK3.6
NL1	trass-lime	TNO1.6
NL2	trass-lime	TNO2.6
NL1	trass-lime+rep.	TNO3.6
It1	hydrat.+pozzol.	PDM1.6
It2	hydrat.+pozzol.	PDM2.6
It3	putty-lime	PDM3.6

On the basis of this common approach, a simple model was used to convert the experimental measurement of Figure 4a to a *modified deterioration profile* where the bulging has been eliminated as shown in Figure 4b. The procedure starts with the comparison between two consecutive profiles n and $(n-1)$: usually the vertical coordinates of n are smaller than the ones of $(n-1)$ except for in the points affected by swelling. In these points the model compares the coordinate of profile $(n-1)$ with the ones of the successive profiles (n) , $(n+1)$; $(n+2)$... until a profile $m = (n+1)$ is found with vertical coordinates smaller than the ones of profile $(n-1)$. The coordinate found becomes the new reference coordinate of profile n in the re-drawing of it. (fig. 4b).

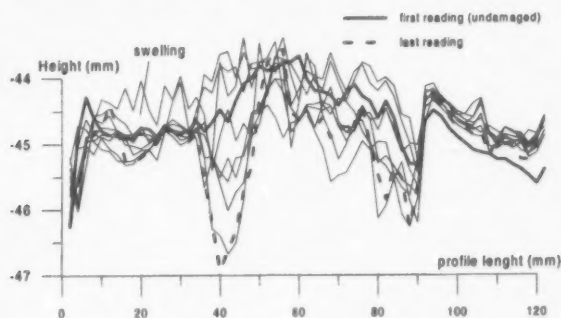


Fig. 4a: Deterioration profiles with swelling

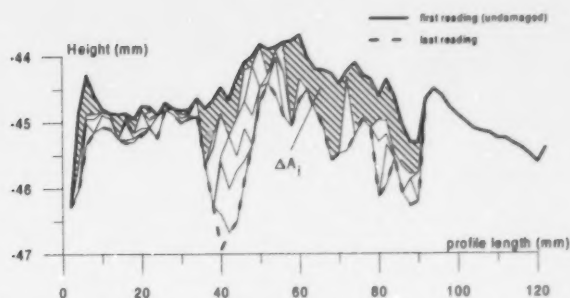


Fig. 4b: Deterioration profiles without swelling

4.1 The damage parameter: a difficult choice

In order to describe the deterioration processes of the materials the choice of an appropriate damage parameter is the first, often problematic, step.

In order to define a way of measuring the decay of masonry and masonry materials, the loss of material from the external surface has been considered an experimental value that seems to be a good indicator of damage increase over time (Binda et al., 1992). The loss of material is described through the changes of the specimen thickness recorded at each measurement carried out at the chosen points: a loss of material happens when a decrease is measured of the vertical coordinate (Molina et al. 1996). From Figure 4b the measure of the area ΔA_i , included between two consecutive readings, can be defined as a *damage parameter*. Therefore, the damage parameter assumed here is the loss L (mm^2) of material with time defined as the sum of each ΔA_i calculated at every step of measurement:

$$L(t_i) = \sum_i \Delta A_i \quad \text{with} \quad i = 1, \dots, r, \dots, 10 \quad (\text{readings}) \quad (1)$$

t_i = reading times, in cycles

In Figure 5 some calculated losses $L(t)$ are shown as an example.

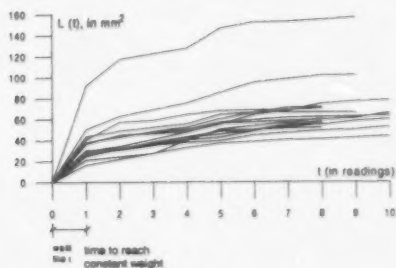


Fig. 5: Diagram of the loss of area over time

5 Experimental results

The aim of the research was to study the behaviour of treated and untreated wallettes during the crystallization tests in order to define the effectiveness of each treatment. The effectiveness was indirectly defined by calculating the damage as described above.

5.1 A case study: untreated wallettes

The procedure presented in the previous Section is here applied only to the untreated wallettes built with different types of bricks and mortar (Table I).

As it was explained in Section 2, measurements were carried out when the wallettes had reached constant mass. This request is a difficult one, since constant mass cannot always be reached because sodium sulphate is a highly hygroscopic salt and therefore water is always trapped inside the material. Nevertheless Figure 6 shows that all the wallettes tend to asymptotically reach constant mass between ten to sixteen weeks except for TNO2.6. This specimen had the worst behaviour suffering delamination of layers to a depth of more than 1 mm. Therefore this specimen was not taken into consideration due to its too different behaviour. Particularly interesting seems instead the behaviour of the KIK wallettes.

As a first hypothesis the wallettes were considered as one population of samples, without distinguishing the different types of materials. In the future the decay of mortar and bricks will be considered separately. Following this choice, the analysis of the data was carried out taking into account the behaviour of the wallette surfaces with time independently from the component materials.

From the results obtained, it appears that deterioration is characterized by two different phases:

1. The deterioration process before the time t_0 to reach the constant weight. As mentioned before, the second measurement was carried out when constant weight was reached. Therefore, the time t_0 to reach constant weight is the same time of the first damage read.
2. The deterioration process over time t with $t > t_0$.

Some damage parameters have been identified and their possible interactions analyzed.

For the *first phase* of deterioration, the parameters identified are:

- a) the initial roughness of wallette surface ΔA_r (Fig. 7). It has in fact been hypothesized by the authors that the initial roughness, typical of stoney, materials could affect the initial damage;
- b) the random variable $(r, v.) t_0$;
- c) the $r, v.$, first damage measure, L_0 at the time t_0 .

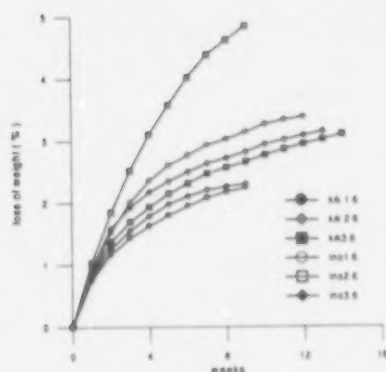


Fig. 6: Diagram of Loss of Weight for $0 < t < t_0$

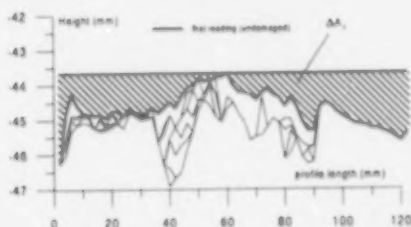


Fig. 7: Deterioration profiles: initial roughness

In the Figures 8a and 8b the damage L_0 is plotted against the time t_0 and the roughness against the damage L_0 respectively. The two plots show that: (i) a weak correlation appears between t_0 and L_0 : a short t_0 corresponds to a low magnitude of L_0 (Fig. 8a), (ii) no correlation appears between roughness and first damage (Fig. 8b). On this subject more investigation is needed.

More in details:

- the TNO wallettes present a higher initial roughness than the KIK, but the first damage does not seem to be influenced by it; the same is for the highest damage L_0 with respect to all others wallettes (Fig. 8b).
- the KIK wallettes seem to present a more clear behavior than the TNO ones. The random variable characteristic of their deterioration process are affected by smaller dispersion indices. Their time t_0 seems to show a bimodal behavior.

The *second phase* describes the propagation of damage over time $t > t_0$. As it appears in Figure 9, the damage process over time $t > t_0$ shows a similar behavior

for the whole population, with the only exception of two profiles of TNO wallets, as above observed.

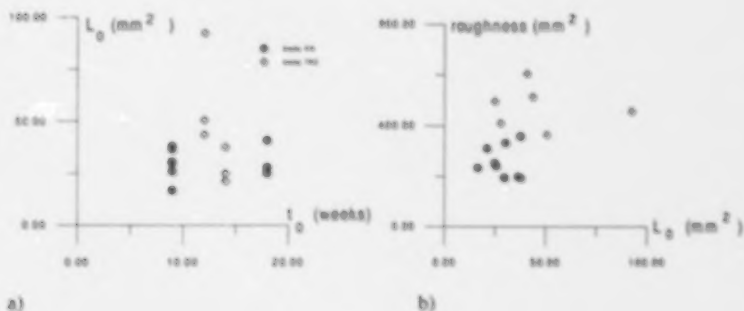


Fig. 8: Experimental data: a) First damage magnitude L_0 versus time t_0 ; b) Initial roughness ΔA_r versus first damage magnitude

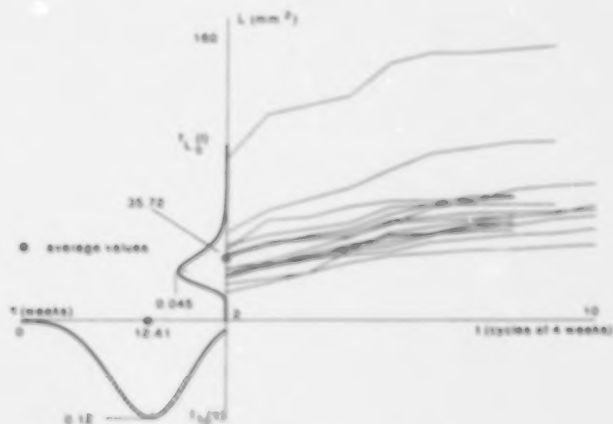


Fig. 9: Deterioration process for $t < t_0$ and $t \geq t_0$; $L_0 = L(t_0)$

5.2 Theoretical approach

From the analysis presented in section 4.2 it appears suitable to approach the two phases of deterioration separately. In fact the two phases are connected with two different deterioration processes: the first is connected with the decay read at constant weight, the second describes the deterioration phenomenon connected with the continuation of the test.

Therefore the aim is to define the behavior of both the random variable t_0 and L_0 and the deterioration process over time t with $t > t_0$ under a probabilistic point of view. As a result:

- The time t_0 can be suitably modelled with a Weibull probability density function (p.d.f.):

$$f_{t_0}(\tau) = \alpha \rho \tau^{\alpha-1} \exp[-(\rho \tau)^\alpha] \quad (2)$$

The shape parameters have been estimated with a computer code involving the method of maximum likelihood. From the fitting a high standard deviation appears, however the curve-fit is satisfied and the average value estimated seems to be a significant value for the time distribution. The curve is plotted in Figure 9.

- The first damage L_0 can be suitably modelled by a Log-normal distribution. The p.d.f. used is:

$$f_{L_0}(l) = \frac{1}{\sqrt{(2\pi\alpha)}} \exp\left[-\frac{[\log(\rho l)]^2}{2\alpha}\right] \quad (3)$$

From the fitting appears a small standard deviation and the curve-fit is satisfied. (Fig. 9).

In the second phase the damage process over time t (with $t > t_0$) shows a similar behavior for the whole population and seems to be independent at the level of first damage L_0 . Therefore the process $L(t)$ at the time $t=t_0$ can be normalized at $L_0 = 0$. In Figure 10 the normalized process $L_n(t)$ is shown. The process $L_n(t)$ can be interpreted like a stochastic process of the r.v. l . At each cycle, the density of $L_n(t)$ can be modelled by a p.d.f. of the type (3) (Fig. 10). The curve-fit of the expected values L_n^e of each p.d.f. seems to represent an average behavior of the whole population (Fig. 10).

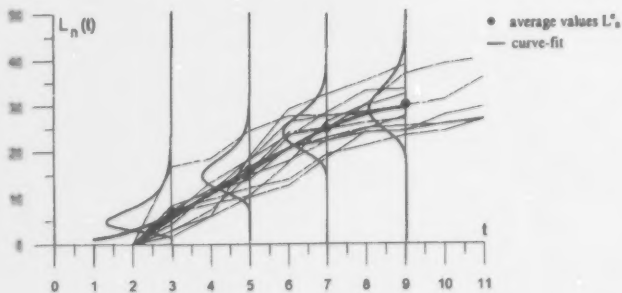


Fig. 10 Stochastic process $L_n(t)$; curve-fit of the average value

Defining $L_n(t)$ like a stochastic process the damage process can be approached like a durability (or reliability) problem: when a significant value $\bar{L}$ is defined for damage the hatched area represents in Figure 11 a reliability function $p_f(t)$ as:

$$p_f(t) = \Pr(L_n(t) > \bar{L}) \quad (4)$$

The function $p_f(t)$ has the characteristic of the distribution function of a random variable that can be suitably introduced as the *service lifetime* $\bar{\tau}$ necessary to reach damage $\bar{L}$, that is:

$$p_f(t) = \Pr(\bar{\tau} > t) \quad (5)$$

A detailed study of the service life $\bar{\tau}$ connected with different values of $\bar{L}$ is needed and represents one of the first steps of the authors future research.

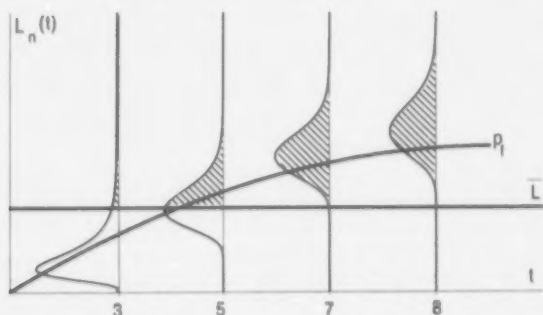


Fig. 11 Reliability function for a significant damage value $\bar{L}$

6 Conclusion

Salt crystallization has caused decay of masonry at all latitudes. Physical and numerical modelling of the phenomenon can be useful to understand its development over time and space.

Deterministic models of the phenomenon for masonry as a composite material are still not available and would be in any case very complex and expensive. The authors have previously proposed a probabilistic model concerning the behaviour of single brick and stone. In order to extend the model to masonry elements they are now processing the data obtained from laboratory tests on wallettes which allowed to: (i) produce a damage mechanism similar to the one occurring in a natural environment, (ii) monitoring of the damage in a non destructive way. The experimental data collected outline the evolution of the deterioration, over space and time, of the mortar-brick composite.

The analysis of the experimental data shows two distinct phases of the damage process. The first phase is characterized by the time t_0 necessary to reach constant weight and by the damage magnitude L_0 . The time distribution can be modelled in a satisfactory way by a Weibull distribution. The magnitude L_0 seems to follow a Log-normal distribution. The second phase is certainly the most interesting one. In fact during this phase the damage process $L(t)$ over time t with $t > t_0$ is practically the same for most of the samples of the population. It seems to be independent of the magnitude L_0 at the initial time of the second phase.

Therefore all the damage curves have been normalized at the initial value $L_0 = L_n(t) = 0$. The normalized damage process $L_n(t)$ has been interpreted as a stochastic process of the random variable l . At every profile measurement the distribution of the values of the l has been modelled as a Log-normal distribution. The average values of the distributions seem to be significantly representative of the average damage of the entire population of the analyzed specimens.

Future aim of the authors is to treat the problem of durability as the probability of reaching a defined threshold of damage $\bar{L}$ in the time $\bar{\tau}$.

7 Acknowledgements

The authors wish to thank G. Cavallini, B. Lubelli, P. Perolari, M. Antico, G. Ghilardi for their help in laboratory and in data processing. The research has been supported by EC XII Commission and MURST.

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51 EVALUATION OF SITE CONTROL TECHNIQUES TO ASSESS THE EFFECTIVENESS OF SURFACE TREATMENTS ON MASONRY

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Abstract

The authors experience based on on-site and laboratory testing has led to the conclusion that the decay of surface treatments has to be studied not only in laboratory but also on site. Therefore, following comprehensive research on the reliability of laboratory ageing tests, it has been decided to use full-scale models to study the durability of water-repellents and consolidants.

Several on-site techniques were applied to study the behaviour of the treatments: laser profilographer, photogrammetry, Karsten tube, pointing mortar tester, coring and measurements of moisture content by powder drilling.

The paper discusses the collected results and compares the different techniques in order to suggest procedures for on-site monitoring of the performance control of the surface treatments.

Keywords: full-scale models, site tests, salt crystallisation, masonry, test methods, water repellents, consolidants

1 Introduction

The evaluation of the performance of surface treatments (consolidants and water-repellents) is a difficult task since the parameters affecting their durability seem to be very numerous (environmental actions, treatment durability, characteristics of the original materials, compatibility between the existing materials and the treatments, etc.) and difficult to measure. Laboratory ageing tests available through existing codes seem to be more useful for the quality control of the materials rather

than give the evaluation of the masonry performance. In fact the laboratory tests in the past dealt with the single masonry components rather than with masonry as a composite. Experimental works carried out by the authors showed that during the decay processes there is a mutual influence developed between mortar and bricks or stones (Baronio et al. 1992; Binda et al. 1992). During the experimental study carried out within an European Community contract (EV5V-CT94-0515) in collaboration with Research Center of Royal Institute for Cultural Heritage, Brussels, Belgium and TNO Building Research, Delft, The Netherlands, the use of both wallette specimens for laboratory tests and full-scale models was proposed to check the treatment durability (IRPA KIK et al. 1996).

1.1 State of the art of the previous research

Baronio and Binda proposed in 1983 a salt crystallisation ageing test using wet-dry cycles based on the Bard test; it can be performed for total immersion as well as for capillary rise. The influence of different salts and different concentrations was detected together with the behaviour of the material under different hygrometric conditions and cycle duration (Binda et al. 1985). The test was also carried out on masonry prisms, and the influence of the mortar and brick properties on the rate of decay was evaluated (Binda and Baronio, 1985). The hygrothermal conditions and cycle duration were chosen as a result of the previous experimental research and of a survey of the environmental conditions in the northern part of Italy (Binda and Baronio 1987).

Different ways of quantifying the decay of the material were subsequently adopted: destructive mechanical tests, sonic tests, evaluation of weight and volume loss. The authors also proposed a procedure for measuring the rate of deterioration of the surfaces applied to the case of protection and consolidant treatments (Binda et al. 1992). A probabilistic model was also implemented to predict the decay of the material based on the laboratory data (Binda and Molina 1990). In 1989 the authors proposed to continue the studies on full scale models which were built in a polluted area of Milan in order to compare the result obtained in laboratory to a natural environment (Baronio et al. 1992).

Finally they have adopted a crystallization test on wallettes proposed by TNO and described in RILEM Recommendation (Rilem127MSTC, 1998).

1.2 Control techniques

Several techniques, some traditional, others proposed by the authors have been applied to test the durability of surface treatments in the short and long term. The aim of the paper is in fact to select the most reliable ones.

The techniques used are based either on the surface control of moisture, water absorption, temperature or in depth control based on the measurement of moisture, treatment penetration and decay.

They can be subdivided into:

- *surface measurements*: Karsten tube (measures the rate of water absorption), pointing mortar tester, (pendulum hammer measures surface hardness of mortar), laser profilographer, (measures the roughness), photogrammetry, thermography, thermocouples. Results of the two last techniques will not be reported here.
- *internal measurements*: powder drilling, coring
- *average measurements through the wall section*: sonic tests and ground penetration radar (result are not reported here).

This paper discusses long term monitoring of brick and stone walls treated in 1992, by using visual and photographic inspection and measurements of the surface decay by a laser profilographer (Binda et al. 1996). A new technique based on photogrammetric surveys will also be briefly mentioned (Bertolo S. 1998).

On recently treated brickwalls where it has not been possible to use the profilographer and a short term response was necessary, other techniques above mentioned above have been applied to check the effectiveness of the treatments in an indirect way. These techniques are partly described in RILEM Recommendation of 127MS Technical Committee (RILEM 127MSTC, 1998) partly by IRPA KIK et al. (1996).

2 On site experimental investigation

The research was carried out on three full-scale models built in 1990 by the authors in collaboration with ICITE-CNR, S. Giuliano M. and ESEM, Milan. Some of the panels of the models were treated in 1992, others in 1996.

The surface treatment procedures are described, and the results of the on-site control are reported.

2.1 Full-scale model description

The models were intentionally built in a polluted area of Milan. Their geometry and exposure were designed to obtain the fastest decay in a natural environment in both the treated and non treated cases. Therefore the possibility of a continuous capillary rise of water inside the walls was realised together with the possibility of producing an accelerated decay due to salt crystallization (Baronio et al. 1992; Binda et al. 1992).

Two types of material were chosen: (1) soft mud facing bricks used for restoration of historic buildings; (2) a sandstone called "Pietra serena" coming from Toscana, and largely used in the central part of Italy. The mortar chosen for all masonry was a lime putty mortar.

Three model types were built, all one storey construction with the principal facades divided into modular orthogonal panels exposed south and west. (Fig.1). Two models, one with sandstone named model A), one with soft mud facing brick (named B) have five pairs of orthogonal panels each (named respectively A1 to A5 and B1 to B5; the third

model (named C), with mixed stone and brick (C1 to C4), has only four pairs of orthogonal panels. The construction of the models ended on September 1990.

In order to study the effect of salt crystallization, an artificial decay is caused in some areas of the walls by introducing a salt solution (Na_2SO_4) into small containers positioned at the bottom of the walls.

2.2 Treatment description

2.2.1 Models A1, A2, A3 and B1, B2, B3

Two epoxy systems especially studied for durable consolidation and protection of brick and stone surface were used in 1992 to treat the external surfaces of panels A1, A2, B1, B2 (Fig. 2) as described in Table 1.

A3 and B3 models were treated in 1992 with an acrylic resin mixed with fine quartz in order to avoid a shining surface. This treatment was applied by brush as a protective film (see Table 1).

B5 was treated in 1996 with two consolidants.

2.2.2 Model C3, C4

Four types of water repellents were used to treat, in January 1996, two panels of full scale model C (Table 2). These products are the same as those used in the laboratory research (IRPA KIK et al. 1996).

All the panel joints were repointed with a less porous mortar in order to avoid water penetration through the joints and allow for a more efficient treatment. Only one portion of C3₂ (C3_{2b}) was not repointed as a comparison. One portion of C4₂ (C4_{2b}) was not treated.

Table 1: Treatments applied to the full scale models A and B

Panel	Treatment	% of resin	Date of application
A1, B1	Cycloaliphatic epoxy resin (EP 150/203)	13%	1992
A2, B2	Aliphatic epoxy based system (EP 222/19)	8%	1992
A3, B3	Acrylic resin (PARALOID B72)		1992
A4	Untreated	/	/
A5	Untreated	/	/
B4	Untreated	/	/
B5 ₁	Consolidant based on ethylsilicate (TEGOVAKON V)		January 1996
B5 ₂	Consolidant based on ethylsilicate (Wacker OH 100)		January 1996

Table 2: Treatments applied to the full scale model C

Panel	Treatment	Denom.	Solvent	% of resin	Date of application
C1	Untreated				/
C2	Untreated				/
C3 ₁	Oligomeric methyl siloxane (TEGOSIVIN HL 100)	WR1	White spirit	10%	January 1996
C3 _{2a-b}	Oligomeric siloxane in water (WACKER 1311)	WR2	Water	10%	January 1996
C4 ₁	Oligomeric methyl-butyl siloxane (WACKER 280)	WR3	White spirit	10%	January 1996
C4 _{2a}	Oligomeric methyl/hexyl siloxane (DRY FILM)	WR4	White spirit	10%	January 1996
C4 _{2b}	Untreated	/	/	/	/

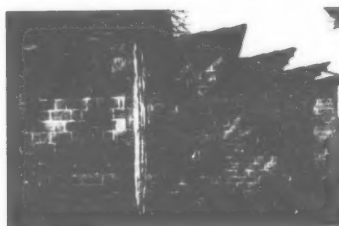


Fig. 1: Full scale models

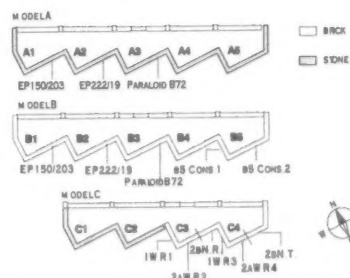


Fig. 2: Surface treatment applied

2.3 Experimental results

The results will be presented separately concerning the treatments applied in 1992 or 1996.

2.3.1 Models A1, A2, A3, B1, B2, B3

The technique of measurement used for these panels was the laser profilograph already described by the authors in (Binda et al. 1993; Binda et al. 1996a). The measurement gives the vertical or horizontal profile of the wall in the chosen positions (Fig.3). Subsequent measurements show how the profile is changing in time due to any superficial decay caused by freeze-thaw, salt crystallization, air pollution, etc. In this way it is possible to measure the material loss with time. The technique is a non-destruction method monitoring the decay.

Figs.4 show an example of the profiles from 4 different measurements which have taken place from 1993 to 1997 (two profiles shown in Binda et al. 1996b). With this type of measurement it is also possible to monitor the decay due to salt crystallization, which proceeds by successive delaminations (Fig.5). Boring of the profile indicates initial detachment of a surface layer which in turn is followed by loss sudden reduction in profile when the layers drops off.

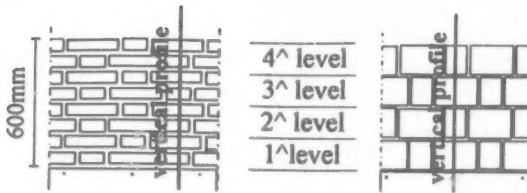


Fig. 3: Vertical profiles and levels

A new technique proposed by Bertolo S. (1998) is based on the use of photogrammetry. A metric camera was placed as near as possible to the wall but at the same time at a distance suitable to include all the deteriorated area in the photograms. A 120mm lens was used to reduce the aberrations. The area of the superimposition of the two photographs was about 1x2.3m. The pairs of photographs chosen for the scanning (using a Digital Scanning Workstation) were selected in order to limit the effect of shade. A high resolution was obtained (pixel size of 13x13micron). A digital model was created: at each area of 0.5x0.5mm corresponds an ideal point defined by a group of three co-ordinates (x, y, z) (Fig. 6). This model the comparison of points measured in two subsequent surveys and to evaluate any loss of material.

Photograms were taken on October 1997 and on April 1998 on the panel A1 up to 500mm from the ground level.

2.3.2 Models C3,C4

The measurement techniques applied to these models were different, due to the impossibility of using the profilographer but also to the necessity of knowing the effectiveness of the treatments in a short time. Soon after the treatment, different points on models C3 and C4 were tested by applying the Karsten tube, the powder

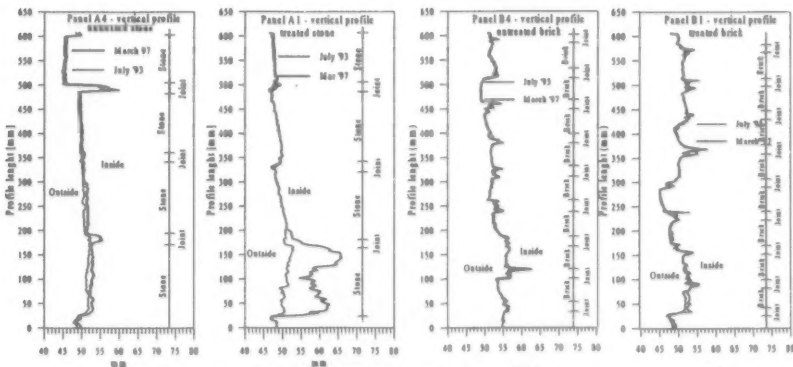


Fig. 4a-b-c-d: Example of vertical profiles (stone and brick, treated and untreated)



Fig. 5: Delamination



Fig. 6: Digital model- treated stone panel - lower part

drilling test and coring. In order to have a quick answer of the durability of the treatments, heavy rain was simulated by directing water to the same chosen points on the panels and then repeating the three tests (Fig. 7). In the Fig 8 the results of the Karsten tube are presented for the untreated part of C4 before and after the rain test (April 1996) and in November 1998 at the same points; a case of a treated panel (C4WR4) is shown in order to compare it to the untreated one.

The moisture content and variation were measured by the powder drilling test. The test was carried out on dry and wet walls at a height of 1.05m (before and after the rain test). Fig. 9a and b shows the results on walls treated with WR1, WR2, WR3, WR4 in the situation dry and wet after the rain test compared with an untreated one. Fig. 9c shows the results of the drilling test on dry and wet walls treated with WR2 with and without repointing. Moisture content increased to more than 10% in the case of treatments WR3 and WR4 after the rain test, while it has slightly changed in the case of WR1 and WR2 with previous repointing. WR2 was not very effective in the case of treatment without repointing (Fig.9c).

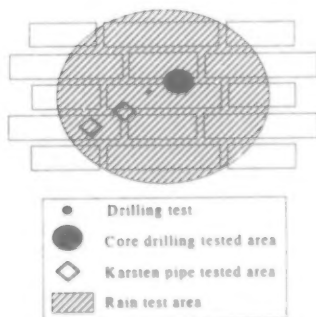


Fig. 7: Location of Karsten tube and drilling test at rain test areas

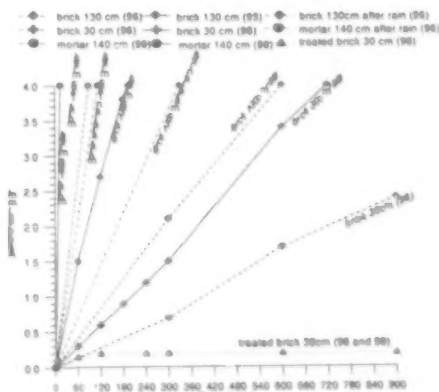


Fig. 8: Karsten tube test on untreated (C4NT) and treated panel (C4WR4)

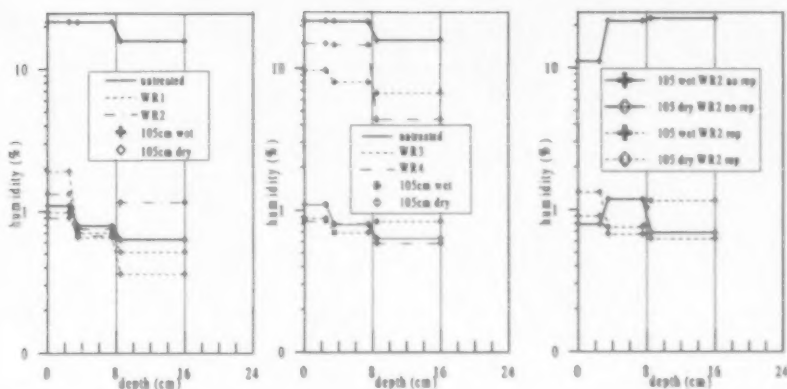


Fig. 9a-b-c: Moisture content (% by weight) based on drilling method results (C3-C4 repointed and treated with water repellent)

The application of the pointing tester clearly showed that this technique can be used for quality control of mortar joints, with the stronger mortars (unless in the future the test is modified for measurements of softer mortars).

The technique can also be used to compare stones and bricks of different hardness; nevertheless it is impossible to compare the same brick or stone in a condition of continuous decay due to the fact that the loss of layers by delamination exposes to the tester a different type of material (IRPA-KIK et al. 1996).

Sonic and radar tests as well as infrared thermography, can be very effective in giving the capillary rise profile in the case of treated and untreated surfaces.

Coring of the treated walls was carried out (IRPA-KIK et al. 1996) and the depth of penetration of the treatment was measured on the obtained cores. The specimens were also put into sealed boxes in order to survey efflorescence and decay (Fig. 10a). Figure 10 shows the penetration of the treatment at the brick mortar interface in the core taken from the panel without repointing (C3WR2_{2b}). This result explains the high moisture content which were not reported (Fig. 9c).

3 Elaboration of the results

3.1 Panels A1,A2,A3, B1,B2,B3

An analysis of the data recorded by the laser profilographer has been carried out in order to understand the difference degradation treated and untreated materials. The difference between the coordinates of the same point between two subsequent measurements allows the loss of material to be calculated (Binda et al. 1996b). Each vertical profile was divided into four levels (Fig. 3) and at each one an average of

about 60 values has been calculated. From the analysis it's clear that the decay in the treated panels is more than in the untreated one; also it is more concentrated at the lowest level and less variable (Fig. 11a-b).



Fig. 10a: Cores into the box



Fig. 10b: Specimen from C3WR2_b

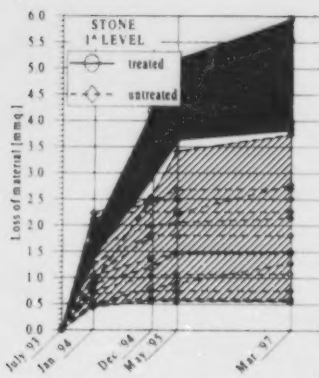
A comparison between the results obtained from photogrammetry and the laser profilographer was also attempted. In Fig 12a-b the same horizontal profile is measured with the laser profilographer and the photogrammetric method. It can be seen that the new method gives more detailed information. Furthermore its advantage over the profilographer is clear. It can be applied far from the surface and it gives three dimensional measurements of the decay. Nevertheless some problems have to be solved: (i) occluded areas (e.g. areas covered by others), (ii) shadow areas, (iii) dark areas, (iv) too homogeneous surfaces (v) object with unfavourable geometry (e.g. spherical objects).

3.2 Panels C3,C4

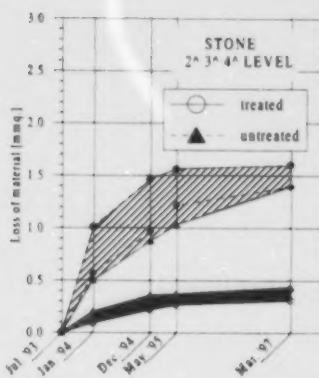
From the results obtained with the Karsten tube, the powder drilling test and the pointing tester some observations can be made: the Karsten tube was able to distinguish between treated and untreated surfaces, but no distinction and comparison could be made between the different treatments in short and long term applications; the treated materials all gave a very low rate of water absorption. Differences were shown in the case of untreated materials before and after the rain test and after two years.

Fig. 13 shows the frequency against the different levels of penetration of the treatment as directly measured on cores sampled after treatments from panel treated with WR1, WR2, WR3 and WR4. It is clear that the more frequent value of penetration occurs between 2 to 4mm.

Fig 14 shows the difference of depth penetration in the case of panel C3 repointed (C3WR2a) and not repointed (C3WR2b) before the treatment at different levels from the ground (40 and 105m) and before and after the rain test (105m height). The apparent higher penetration in the case of no repointing is actually due to the penetration of the treatment between the contact surface brick-mortar as it is shown in Fig. 10b.

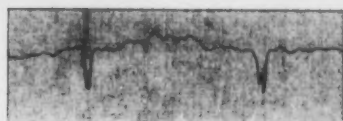


(a)

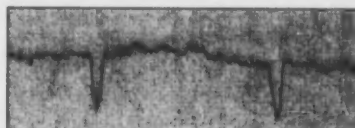


(b)

Fig. 11a-b: Comparison of surface loss of material between and untreated walls



(a)



(b)

Fig. 12a-b: Comparison of surface loss of material between profiles obtained by:
 (a) laser profilograph
 (b) photogrammetry

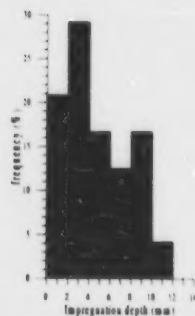


Fig. 13: Impregnation depth against frequency

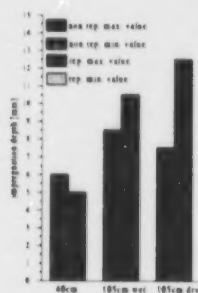


Fig. 14: Comparison of impregnation depth

Table 3 gives the average moisture content from two measurements by powder drilling at 30mm and 80mm depths. Three comments can be made:

- (i) an untreated wall with repointing tends to absorb water in a similar way as a treated one without repointing,
- (ii) a treatment carried out without repointing the wall can have very little effect and this fact is probably due to the water penetration at the brick-mortar interface and through the mortar joints if they are too porous,
- (iii) in the case of very wet areas (see 40cm height) the efficiency of treatments tends to be lower, unless a drying process cannot be realized before treatment.

Table 3: Medium value of moisture content (% by weight)

Height	Brick					Mortar		
	C3WR1 repointed	C3WR2 repointed	C3WR2 not repoint	C4WR3 repointed	C4WR4 repointed	C4NT repointed	C3WR2 repointed	C3WR2 not repoint.
105cm wet	1	1	15	8	12.5	22	0.5	12
105cm dry	1	1	1	1	1	1	0.5	0.5
40cm dry	5	3	3	1.5	2	2		

4 Conclusions

From the experience carried out on the detection of treatment efficiency some conclusions can be drawn:

- efficiency and durability of treatments cannot be directly measured, but indirectly detected through the measurement of depth of penetration, moisture content and surface decay;
- the Karsten tube test can be used in short and long terms to distinguish treated from untreated surfaces but is not able to distinguish between different treatments in the short or long term;
- the drilling test method can give in the short term the possibility of comparing different treatments through the measurement of moisture penetration and diffusion; it is also a slightly destructive method and therefore it cannot be used as a continuous monitoring technique;
- coring gives a direct inspection in short and long terms but it is a destructive method and cannot be used as a continuous monitoring technique;
- the measurement of surface profiles can be very effective as long term monitoring technique; it is the only technique at present which can accurately measure the decay process of a material;

5 Acknowledgements

Authors wish to thank E. Gaini, C. Casana, G. Cavallini, M. Vivian, P. Perolari, G. Ghilardi, for their help in laboratory and on site. The research was carried out with the support of EC XII Commission and CNR.

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52 CHARACTERISATION OF MATERIALS USED IN THE REPLACEMENT OF SCULPTURES IN HISTORICAL MONUMENTS

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Abstract

The objective of the present study is to appreciate the aptitude of various materials to imitate stone for the realisation of a moulding. After having referenced the main defects which appear on the copies, the moulding of a statue representative of the main difficulties encountered on the statuary of monuments was made with each of the materials. Four mortar formulations (pre-formulated or hand made) as well as two resin-based products (polyester resin and resin mortar) were tested and characterised, in order to establish the connection between defects and implementation. The first results indicate that the quality of a moulding greatly depends on product use, particularly in the case of hydraulic mortars. Moreover, the risks of emergence of defects can be reduced with a refined formulation.

Keywords: copy, mortars, moulding, resin

1 Introduction

Since the first half of the 19th Century, pollution has led to an increase in the degradation process of monuments, further aggravating the impact of the climate. When sculptures of a high aesthetic quality thus suffer from being exposed to various atmospheric agents, the people in charge of historical monuments resort to having them taken down and sheltered away.

The question of the replacement of sculptures on monuments which would otherwise be defaced by their removal, is therefore raised. The replacement of deteriorated sculptures was traditionally accomplished with stone, thanks to the talent of sculptors or stone-cutters. But we have recently seen mentalities evolve towards a concern for maximum integration, and, in environmentally poor conditions, indeed even in the case of ruins, a new practice, the replacement of the originals with "reproduced stone copies" obtained through moulding has developed and meets with a

growing success. In this way, within the framework of the actual restoration of the occidental façades of the cathedrals in Reims and in Rouen, a number of high-quality sculptures are being taken down, cleaned, treated and sheltered in museums to be replaced on site by mortar mouldings which are perfectly integrated to the site.

A certain number of problems remain however. Thanks to the observation of many mouldings exhibited in places like the Tuileries Gardens in Paris, at a motorway rest area in the South of France (the Lozay themed rest area — a roman itinerary in Saintonge), or of mouldings already installed on the Reims and Rouen cathedrals, the main problems encountered on a moulding were revealed.

Six mortar and resin formulas likely to duplicate the aspect of the stones from two monuments (where test sample sites were established) were therefore studied in association with two types of mineral patinas (acrylic and biomineralisation).

The study focused on one hand on the characterisation of the materials which we tried to relate to particular defects and on the other hand on the durability of the products. Because of the difficulty in dissociating the influence of the implementation of the products on their durability, all of these factors was examined independently. The tests required a very close collaboration with restoration workshops, the companies offering and implementing their products, and the assistance of a number of laboratories.

To characterise the aptitude of the products at implementation, the moulding test of a sculpture was carried out with each product, providing the means to assess the advantages and disadvantages of each "product / implementation" couple.

Each product is characterised with a physical/chemical test in order to make sure of an adequate compatibility between itself and the stone.

Last of all, product durability is evaluated with ageing tests. Three categories of tests are carried out: ageing on natural sites or in a climatic ageing room and resistance to the frost cycles.

2 Results of the survey on the conservation condition of ancient copies

The main problems affecting the aesthetics and the adequate ageing of a moulding were brought to light through the observation of a number of copies resin-based or made with hydraulic mortars.

The problems which were observed on the mouldings can be classified into two categories : these problems related to the implementation of the material during the realisation of the moulding copy and those relevant to the ageing of the moulding product components.

2.1 Implementation-related defects

In this first category were grouped the defects associated to implementation techniques as well as those more closely related to the rheological product properties in terms of fluidity and viscosity.

Blowings or poppings, the presence of isolated relatively hemispherical cavities. Depending on their size and importance, it may be necessary to fill them up. **Single-pour materials** are more likely to produce surfaces with blowings. Product

formulation and implementation are two criteria that come into the blowing occurrences. A bad pour, an unsuitable vibration or a mould which has been inadequately cleaned or which is not very wettable can provoke a concentration of air bubbles on the surface of the moulding. Moreover, an unsuitable granular composition (lack of fines, too much sand) or insufficient workability are also determining factors in the development of blowings.

Bleeding appears in the form of irregularly-eroded zones with small channels where the sand seems to have been washed out. This type of defect, associated with the use of *hydraulic products*, generally provokes colour variations and increases surface porosity. The causes of bleeding are frequently the use of a hydraulic mortar which is either overdoled in water or lacking in fine elements and sand. It may also be provoked by punctual excessive vibration or the presence of water in the mould.

Pattern cracking designates the occurrence of fine cracks near the surface on *hydraulic mortar* mouldings which combine into a relatively fine and intertwined network. Superficial pattern cracking does not structurally endanger the moulding, but it fills with dust and gives the moulding an aged look.

Colour variations are generally related to the execution of touch-ups on the mouldings. Touch-ups carried out with *hydraulic mortars* are more sensitive to this as their composition is different to that of the mortar of the moulding itself (sieving of the product) and their colour as well as their porosity are therefore different.

Lack of material is caused by an improper repartition of the product within the mould (sealed vent) or through the use of a product which is too viscous. In the smaller cases of lack of material, this can be touched up, otherwise, the moulding has to be poured again.

2.2 Material ageing-related defects

This category groups the main imperfections and deterioration which can appear in the open air as the mouldings age.

Cracking is not very aesthetic and can be dangerous if the moulding contains frameworks which are liable to rust. A low flexibility material will be more likely to crack under the provoked inner stresses particularly when confronted to thermal and water-related dilatations.

Efflorescence is characterised by the formation of white-coloured salts (generally calcium carbonate CaCO_3) on the surface of the moulding.

Dirt is provoked by particles suspended in the air which are deposited and transported by water. Dust particles can adhere to the surface because of the micro-roughness of the surface as well as with the capillary forces due to the water the surface porosity retains.

The development of micro-organisms depends on the ecological conditions (nature of the substratum, climatic and biotic factors). A damp site in the shade will be favourable to the development of fungus for example.

Atmospheric attacks are notably provoked by acid rains resulting from the emission of polluting gases (SO_2 , NO_x et CO_x) in the atmosphere. In contact with the *cement hydrates* (portlandite in particular), aggressive agents can form expansive compounds (the oxidation of sulphur dioxide into sulphuric acid can provoke a transformation of portlandite into ettringite with expansive properties) can induce cracking or lead to the dissolution of certain hydrates (action of carbonic acid on portlandite).

The action of Ultraviolet light may significantly make *resin-based* mouldings turn yellow, thus impairing the aesthetics of the moulding.

3 Description of the tested products

The choice of the materials used for the realisation of the mouldings was focused on six products representing the most frequently-used product families. Four of these are hydraulic mortars. The last two products are a polyester resin including 50% marble powder and a 'mortar-resin' (an acrylic resin and semi-hydrate-alpha plaster) also including a mineral load.

Ready-mix mortars are materials that are industrially designed, packaged in bags and ready to be mixed. Hand-made mortars designate the materials that are formulated on site. Last of all, pre-formulated mortars are materials that are specifically designed on request in a laboratory and hence with a distinct formulation.

The main characteristics of the mentioned products are shown in the following table.

Table 1: Description of the 6 tested products

Products	Ready-mix mortar	Ready-mix mortar with fibre	Hand-made mortar	Pre-formulated mortar	Polyester resin	Resin mortar
	MPE	MPEF	MA	MP	RP	MR
Nature of the aggregates	Limestone and silica	Limestone and silica	Limestone and silica	Limestone and silica	Marble powder	Silica
Cement	CPA-CEM I and filler	CPA-CEM I	CPI-CEM II	CPA-CEM I	Synolite polyester resin	Acrylic resin, semi-hydrate alpha plaster
Additives	Super-plasticiser and defoamer	Super-plasticiser, defoamer and fibres	Super-plasticiser, colour additives	Silica in suspension	Silica gel, colour additives	Retarder
Water-Cement ratio	0.5	0.5	0.9	0.6	/	/
Density	2.1	2.1	1.8	2.0	1.3	1.8

4 Experimental protocol

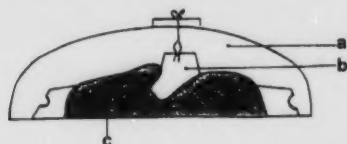
A number of evaluation and characterisation tests were carried out in order to assess each product's aptitudes.

4.1 Implementation test

The products' aptitude for the realisation of a moulding was determined by the implementation of a moulded copy of a statue. "Enfant à la coquille (Child with shell)" (figure 2) was chosen as 'test' sculpture. Sculpted from Estailades stone and 80 cm high, this statue is representative of the main difficulties which can be encountered during the realisation of the moulding of a sculpture from the statuary of a monument, although its size is considerably smaller (The statue of Abraham of the southern gate of the west façade of the Notre-Dame de Reims cathedral is about 3 metres tall). But

the difficulties such as the presence of feather-edge, of back feather and the tortuous shapes where product implementation is delicate, are similar.

The moulding of the test piece is carried out according to traditional moulding techniques, following the principles for the execution of piece moulding (figure 1), i.e. a reusable plaster mould made with a number of parts. The parts of this type of mould adjust to one another and are maintained with a cover.



a. cover
b. small cover attached to the master cover with a string and a ring
c. moulding or original

Fig. 1: Piece moulding

The taking of the print on the original is carried out using two silicones. The cover is in plaster. Small plaster covers provide the means to maintain the silicone inside the cover and make the removal operation from the mould easier. Vents, set in the areas where air is the most likely to be trapped, make its evacuation from the mould easier when it is being filled and they also provide the means to control the access of the product in the whole volume of the mould (figure 3).

This mould is to realise the 6 copies with each of the products (figure 4).



Fig. 2: original test piece
(sizes : base 30x40 cm,
height 80 cm)



Fig. 3: mould of the statue
(1= silicone mounts; 2=small
cover; 3=vents)



**Fig. 4: test copy in
MPEF**

- Hydraulic mortars are poured in the mould. The mould is filled with the product using gravity for its positioning. It is possible to use a dip tube to limit the height from which the product will flow. The material is always vibrated (manually or with a vibrating needle) in order to facilitate the access of the product in the mould and to provoke the suppression of the air included in the mortar.

• Resin-based products are stratified. The stratification technique consists in the application of a "gel-coat" on the different parts of the mould and therefore require to work on an open mould. The applications are carried out in various successive coats (until a 5 to 6 mm thickness is reached), the last applications being rigidified through the use of fibre. Once the "gel-coat" has been applied, the mould is shut and "blocked" (Fibre bands impregnated with resin are applied at the seams in order to join the different parts of the mould together), then filled. The vents must therefore be blocked for the realisation of two resin-based mouldings, realised using the stratification technique.

After the realisation of the 6 mouldings of the test statue, the implementation defects were noted (table 2). The main defects observed are the presence of bubbles in varying sizes and repartition, bleeding, colour variations at the seams and the vents (partly due to the loss of constitutive water, connected to leaks of laitance, towards the outside of the mould during the product vibrating operation during implementation).

Table 2 : main defects observed on the copies of the statue

Products		Ready-mix mortar	Ready-mix mortar with fibre	Hand- made mortar	Pre- formulated mortar	Polyester resin	Resin mortar
		MPE 1 mm	MPEF 2 mm	MA 1 mm	MP 1 to 2 mm	RP	MR
Blowings :	average Ø					-	1 to 2 mm
	Repartition	+ to ++	++	++	++ to +++	-	-
Bleeding		+	+	++	-	n.a.	n.a.
Colour variation at the vent and at the seams		+	+	+	+	-	-
Presence of cracks		-	-	-	-	-	+
Lack of product		-	-	-	+	-	+

Key : - punctual ; + a few zones ; ++ wide zones ; +++ larger zones

• Among all the tested products, mortars are the most likely to have a surface presenting **blowings**. Figures 5 and 6 display zones in the test statue where the blowing of the copies is very important (bottom part of the shell held by the child). The blowing is mainly present in the zones where the air remains trapped in the mould, i.e. at the folds or in the feather edges. Although many vents were located in those places, and despite all the care taken during the realisation of the moulding, numerous bubbles clung to the surface of the silicone. For mortars, the relatively weak MPE blowing is probably due to the use of a dip tube when product was poured.

• The presence of **bleeding zones** is to be attributed either to a lack of fines in the product or to excess water (hand-made mortar).

• Colour **variations** are located at the vents and the seams of the mould. They are provoked by the water movements. Actually, needle vibration of the product in the mould provokes a water migration to the vents and the seams of the mould. In those places, the mortar is hydrated differently (variation in the Water/Cement ratio), which has an incidence on the colour of the material (figures 5 and 6).

- **Fine cracks** were observed on one of the mouldings realised using stratification. They were created during the manipulation of the two shells of the mould, during assembly, before the pour. Stratification moulding realisation requires a great deal of attention during the fitting of the mould, because the smallest flexion of the mould can provoke ruptures in the stratified coat (which is only 5 to 6 mm thick).
- **Lack of material**, observed in two cases, correspond to a part of the mould which is difficult to access (a finger of one of the hands). The absence of that element is due to a problem in the implementation of the material in the case of the gel-coat and a viscosity problem in the case of the MPE mortar.



Fig. 5: test piece
1=blowings ; 2=darker zone (vent)



Fig. 6: test piece 1= important
blowing around vent; 2=colour
variation at seam

4.2 Physical properties of the products

The four formulations of hydraulic mortars were characterised. The results are presented in the graphs below (figures 7 to 14). The characteristics of the two stones in which the statues from Reims and Rouen cathedrals (Courville stone called PC and Vernon chalk called PV) are carved are presented in the different figures below, in order to compare the properties of the moulding mortars with those of the replaced stone. These two stones were used in particular for the construction of the buildings where the natural ageing sites are located (Reims and Rouen cathedrals).

• The hydric properties of moulding mortars are shown below (figures 7 and 8). The tested moulding mortars have a water accessible porosity close to that of the stones. However, the capillary absorption coefficients of hand-made mortar and pre-formulated mortar are far more important than that of both stones.

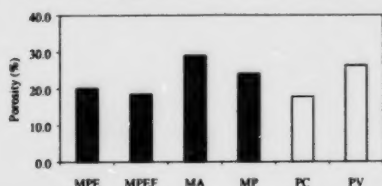


Fig. 7: water accessible porosity (RILEM I.1 recommendations)

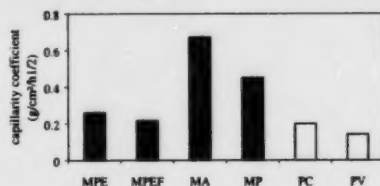


Fig. 8: capillary absorption coefficient (RILEM II.6 recommendations)

• The mechanical properties of the moulding mortars (figures 9 and 10) are determining factors to obtain an adequate durability. The hand-made MA mortar has a low mechanical resistance, due to a high mix rate (E/C=0.9), but it also shows greater elasticity.

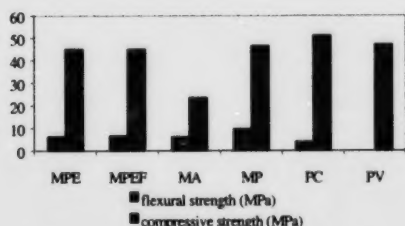


Fig. 9: flexural and compressive strengths after 28 days (NF EN 196-1)

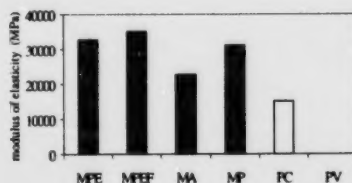


Fig. 10: dynamic modulus of elasticity (RILEM III recommendations)

The relative flexibility calculation ($R_{\text{compression}}/E$) shows that despite different mechanical properties, the materials are relatively equivalent in terms of flexibility (figure 11). Only the pre-formulated MP mortar is likely to bear more important stresses for an equivalent deformation.

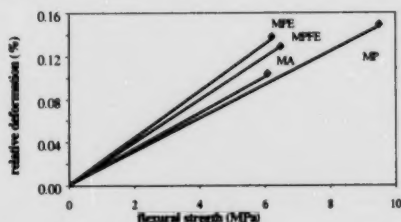


Fig. 11: calculated relative flexibility

The surface hardness (figure 12) of the four mortars are similar, those of ready-mix mortars being more important. This difference in surface hardness can have as an effect a difference in the nature of the laitance.

Thermal and water-related dilatations (figure 13) are especially important with regard to stone's dilatations in the case of the pre-formulated mortar. It will be more sensitive to variations in the climate, and can lead to a progressive damaging of the cement paste due to the stresses provoked by thermal and water-related flexibility.

The shrinkage (figure 14) of the mortars is weak. Leaving aside ready-mix mortar with fibre, the early shrinkage (or endogen shrinkage) is moderate (-40 to -50 $\mu\text{m/m}$). Additionally, shrinkage after 28 days of the four mortars does not exceed 1000 $\mu\text{m/m}$ in all cases.

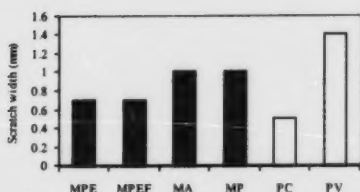


Fig. 12: surface hardness (scratch width) (RILEM IV.1 recommendations)

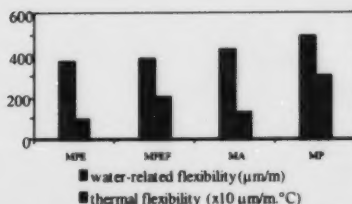


Fig. 13: thermal and water-related flexibility (RILEM VI.3 et II.7 recommendations)

The rates of soluble salts (SO_3 , NO_3^- et Cl^-) contained in the mortars (figure 15) are weak. As the mouldings must replace elements of the statuary, it was necessary to control the absence of risks of migration of soluble salts from the mortar moulding to the constitutive stone of the building.

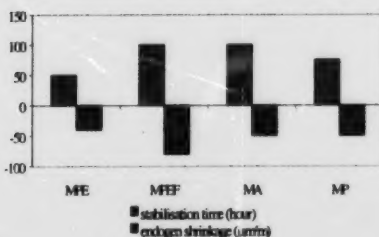


Fig. 14: endogen shrinkage and stabilisation time (IS/LP/E/23 procedure)

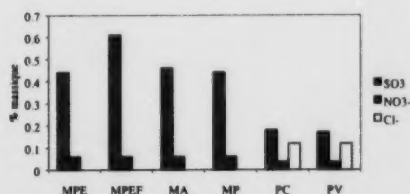


Fig. 15: soluble salt dosage (DOC 13/83 Italian standard)

5 Discussion and conclusion

The present article gives the first results in the characterisation of various materials used for the realisation of copies of sculptures through moulding and it recapitulates the most frequently obtained defects on a moulding carried out with each one of these products.

It appears that the quality of a moulded copy depends on many parameters, such as the nature and the properties of the material, but also including its implementation.

It is undeniable that hydraulic mortars provide the most faithful restoration of texture and appearance of the stone, and that, without the need to patina the moulding. But quality in the implementation of hydraulic mortars is essential. The risk of blowings can be reduced through careful mixing and limitation in the product pour height (for example, through the use of a dip tube which is progressively drawn upwards during the pour), also reducing segregation risks. The mould should not be over-wetted because this can also favour the occurrence of pattern cracking.

The stratification technique, applied to resin-based products can minimise the risks for defects, particularly the presence of bubbles on the surface of the statue and bleeding. Touch-ups, which brings about colour variation problems and the question of durability, are also less numerous and almost invisible.

However, we can observe that through a relatively precise formulation of the mortars, it is possible to minimise the risks of defects. Excess water in the product can induce risks of bleeding and of segregation of the material. But numerous families of additives can improve product quality for the realisation of the copy (as well as its durability). The use of a super plasticiser can improve the implementation of the material in the mould while maintaining a weak water-accessible porosity. An defoamer agent can reduce the quantity of air included in the mortar, and associated to an adapted implementation (avoiding excess vibration which concentrates air bubbles in contact with the mould) can lead to a better-quality surface aspect. The addition of silica in suspension reduces the risks of bleeding. The admixture of additives to ready-mix or pre-formulated mortars which was the object of a in-depth study can help limit defects in related to traditional mortar.

In order to complete the tests which have already been carried out, a set of tests the purpose of which is to assess the durability of the six products is on-going. In particular, these tests consist in the ageing of samples, on two natural sites (Reims and Rouen cathedrals). A follow-up of climatic conditions and of pollution (SO₂, NO_x and black smokes) is implemented to keep track of the behaviour of the samples, as well as the speed with which they become dirty. At the same time is being carried out an ageing simulation in a climatic ageing room, as well as a frost cycle resistance test. We will thus have the possibility to compare the results of the accelerated ageing tests with those of the natural sites.

6 Acknowledgements

This study was financed by the Cercle des Partenaires du Patrimoine association thanks to sponsorship of Lafarge. We thank the laboratory Lafarge Produits Formulés, the moulding (and restoration) workshops J.L. Bouvier, Mérimod et Legrand, the Etudes et Préfabrication Industrielle, Jesmonite Limited and the Société de Décoration et de Moulage companies as well as the LERM which carried out part of the analyses presented here, for their active participation.

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53 PERFORMANCE EVALUATION OF WATER REPELLENTS FOR ABOVE GRADE MASONRY

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Abstract

Water penetration across brick masonry exterior walls is a problem which asset managers, building owners and construction professionals must deal with frequently. Water repellents are often used in an attempt to resolve water penetration problems in certain remedial applications. Unfortunately the information available to building owners and construction professionals regarding the use and evaluation of water repellents is limited. Little practical information exists to assist in the selection of such products and in the evaluation of their performance and durability. As part of a CMHC research project, an evaluation of such products was carried out, exposing the problems of performance evaluation of water repellents for above grade masonry. Six series of tests were conducted using a modified ASTM E-514 procedure and water uptake tube methods to evaluate the performance of five commercially available masonry water repellents. The results of the study have indicated substantial improvements in the resistance to water penetration upon application of these products to clay brick wall assemblies. The data accumulated during this study may have also revealed a trend indicating an increase in the rate of water penetration from the time of product application (i.e.; reduced repellency), necessitating product reapplication after a given time interval. Readers are cautioned in the interpretation of the test data without additional information pertaining to the vapour diffusion characteristics of the applied products and durability concerns.

Keywords: water repellent, masonry, performance, durability, water penetration

1 Introduction

Water repellents are too frequently used as a quick fix in an attempt to resolve water penetration problems for certain remedial applications on brick masonry exterior walls. Water repellents can be effective in reducing water infiltration in certain applications if applied following a thorough diagnosis of the underlying problems and providing that the wall is properly maintained. However, the application of a water repellent is not a miracle remedy for water penetration problems and proper selection of an appropriate product to meet a project's particular needs is essential to secure the performance sought. Unfortunately, information regarding the use and evaluation of water repellents is limited. Little practical information exists to assist in the selection of such products and in the evaluation of their performance and durability. The research project presented in this paper explains and discloses the problem of performance evaluation of water repellents for above grade masonry.

The primary objectives of the research work were to independently evaluate the performance of various water repellents on a comparative basis and to develop practical methods to evaluate their effectiveness in reducing water penetration. In addition, the intent was to monitor short and long term performance of repellents after exposure.

2 Project methodology

In order to address these objectives, three test procedures were used for purposes of evaluating water repellent performance. These tests included a modified ASTM E-514 test procedure and RILEM water uptake tube tests, to evaluate water penetration performance and air infiltration tests to evaluate air leakage characteristics of panel assemblies.

Another very important aspect in the evaluation of water repellents, not covered by this study, is vapour diffusion performance. This essential facet must also be considered in the selection of a water repellent and is currently being investigated.

2.1 Test chamber

A specially designed test chamber was built and six brick wall specimens were erected as shown in Fig. 1 below. Five of the panels were treated with different water repellents for comparative testing. The test chamber was positioned so that the specimens maintained a permanent southern exposure. A series of six tests of both types were undertaken between the months of May to November 1996.

The ASTM E-514 test method provides specific guidelines for a water penetration test chamber system. However, in order to be able to address all of the project's criteria, a custom designed test chamber was constructed. The test chamber was designed to receive the six brick specimens measuring each 1.57 m in height by 1.42 m in width. Refer to Fig. 1.

With particular attention to air tightness, all chamber joints were sealed from the interior and special care was taken to ensure a continuous air seal. The chamber was provided with an airtight access door, interior lighting, a water collection system

and an exterior pressure reference. Refer to Fig. 2 for a schematic representation of the test set-up.

2.2 Sample preparation

The brick wall specimens were erected by professional masons. Standard clay brick with nominal dimensions 230 mm x 70 mm x 88 mm and pre-mixed mortar was used to construct the samples and were erected over a consecutive two day period under conditions similar to normal site conditions. Particular attention was taken to ensure consistency in the laying of the bricks and mortar preparation.

The test panels were each treated with a different product specially developed for masonry applications. One of the six panels remained untreated and served as a control panel. Refer to Fig. 1 for the identification of test panel assemblies.

2.3 Choice of water repellents

Five water repellent products were used in this research project. Their selection was based on the following criteria:

- Reputability and sound technical support
- Product selection and presence in the market place
- Commitment to product research and development
- Willingness to participate

The products evaluated are identified generically as:

- Panel A: 40% Silane (solvent based)
- Panel B: Polysiloxane blend (solvent based)
- Panel C: Silane/polysiloxane blend (water based)
- Panel D: Siloxane/silane blend (water based)
- Panel E: Elastomeric waterproof coating
- Panel F: Control Panel (no coating)

The products were applied by their respective manufacturers in accordance with their written recommendations.

2.4 Testing methodology

A description of the various test methods utilized for this research project is presented in the following sections.

2.4.1 Modified ASTM E-514 test method

The ASTM-E514 test procedure was developed for laboratory testing to evaluate water penetration and leakage through masonry. Several modifications to the standardized test chamber and procedures were undertaken to accommodate all of the proposed tests and to permit weathering to simulate field conditions. In order to accelerate the tests, interchangeable spray racks were built for simultaneous testing of up to three samples.

In accordance with ASTM E-514, a differential test pressure of 500 Pa and a water application rate of 138 l/hr per square metre of wall area were utilized. A minimum sample area of 1.11 m² was exposed to the test and a minimum sample height and width dimension of 1,219 mm was also respected. As specified by the test

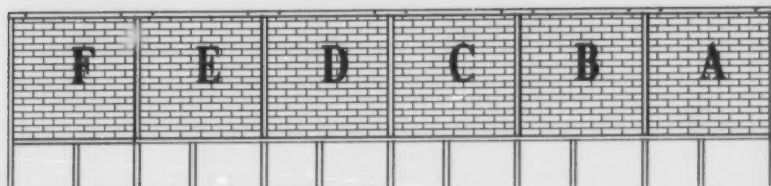


Fig. 1: Identification of test panel assemblies

procedure, testing was carried out continuously for a four hour period. Water which penetrated the wall assemblies under test conditions during this time was collected and measured. The test setup and its various components are shown in Fig. 1 and Fig. 2, respectively.

2.4.2 RILEM water uptake tube test method

The testing apparatus consists of a graduated pipe-like apparatus designed for measuring water uptake on vertical surfaces (refer to Fig. 3). Several versions of the tube are available and vary in construction depending on their supplier. A tube manufactured by ProSoCo was arbitrarily chosen for use in the testing.

The tube is graduated from 0 to 5 ml with each graduation representing an increment of 0.5 ml. The tube was fixed and sealed to the masonry surface under evaluation and filled with water. The quantity of water absorbed by the surface of the masonry material over a specific period of time was recorded and used to characterize the walls' repellency. Testing was conducted in accordance with RILEM II.4 test method, developed by the International Union of Testing and Research Laboratories for Materials and Structures or RILEM (Reunion Internationale des Laboratoires d'Essais et de Recherches sur les Matériaux et les Constructions). At most four readings were carried out per wall specimen; two readings at mortar joints and two readings on the face of the brick unit.

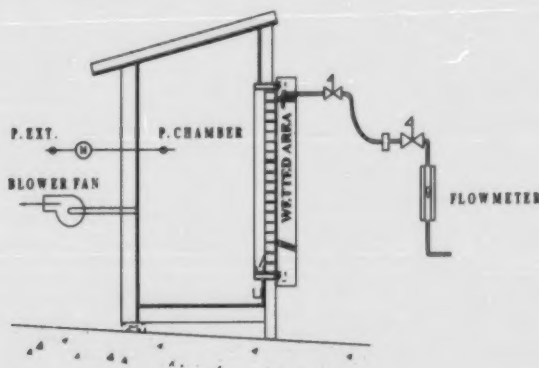


Fig. 2: Modified ASTM E-514 test apparatus

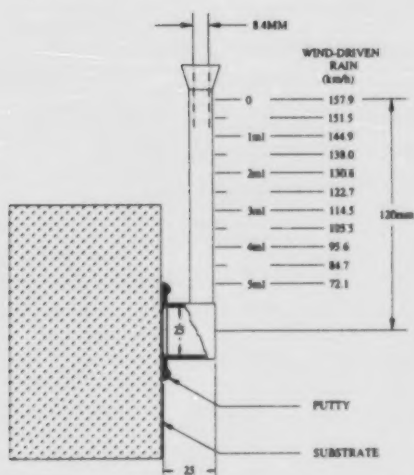


Fig. 3: Water uptake tube test apparatus

2.4.3 Air infiltration test

Air infiltration tests were undertaken with a separate air chamber designed to fit on the interior side of the wall specimen and clamped to the chamber framing for testing of individual specimens. The chamber and test set-up are shown schematically in Fig. 4. The rate of air leakage through each of the specimens was determined through the use of calibrated orifice plates.

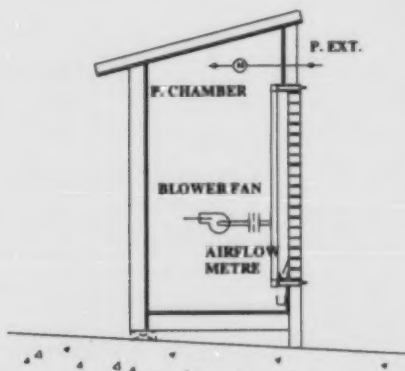


Fig. 4: Air infiltration test apparatus

3 Analysis of results

Given the preliminary nature of the experiments and the limited available data, comments presented in the following sections are solely for general discussion purposes. It is not the intent of this paper to draw any definite conclusions from the data. As such, the basis of the following discussions will be that of establishing possible trends and to obtain an understanding of the performance of products under evaluation and the limitations of the test methods employed.

3.1 Modified ASTM E-514 testing

The relationship between accumulated water (which passed through the specimens) and elapsed time of the test were examined.

The rates of water penetration through the untreated masonry panels ranging from 5,176 ml/hr to 12,265 ml/hr were noted once steady flow conditions were obtained. Three generalized bands of flow rates: 5,000 to 6,000 ml/hr, 11,000 to 12,000 ml/hr and 9,000 ml/hr for panels A and E, panels B, C, and D, and F, respectively were recorded.

Upon first examination of the data, the relationship between accumulated water and elapsed time appeared to be quasi-linear after an initial transitory period, for most of the test specimens (coated and uncoated). However, this transitory period appeared to be very short for untreated panels where an almost true linear relationship was attained after approximately one half hour of testing. For panels treated with masonry coatings, this transitory period was estimated to be approximately two to three hours prior to attaining steady flow conditions.

The variation in water penetration rates for the untreated panels may be attributed to the inherent anisotropic nature of the materials and slight imperfections introduced during the assembly of the test panels.

Application of the various masonry coatings to test panels A through E resulted in a dramatic decrease in the rate of water penetration for all coated panels; as was demonstrated by a series of tests which were conducted 35 days after product application. Generally, a reduction of 44% to 99% from the initial water penetration rates of the uncoated panel assemblies were obtained.

A consistent drop in the performance of masonry panel assemblies was noted for the water penetration test results conducted over the seven month testing period. The performance of panel assemblies was plotted with respect to time and is presented in Fig. 5. The gathered data may indicate a trend of decreasing performance over time. However, additional data is required to fully support this claim.

Based on the limited test data, the rate of percentage decrease in performance over the initial base performance of the uncoated panels was estimated for the various masonry water repellent materials. Based on these rates, the performance decrease and the projected reduction in water penetration for the assemblies were estimated for periods of three, six and nine months, and one, two and three years following application of the masonry coatings. Based on the estimated rates of performance decrease, several of the applied coatings may require reapplication within two years in order to maintain a suitable level of effectiveness.

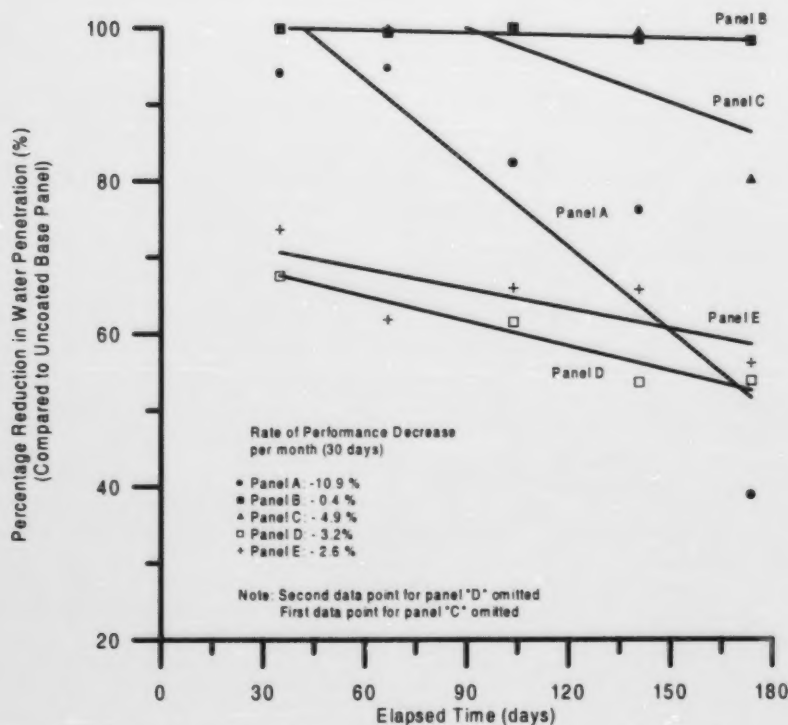


Fig. 5: Estimated coating performance vs. time

3.2 RILEM Water uptake tube tests

Detailed data representing the time required to empty the graduated 5 ml volume of the test tube was recorded for predetermined locations on the surface of panel assemblies A through F. The data retrieved from these initial tests was observed to exhibit variability similar to that noted for the modified ASTM E-514 tests. In particular, substantial differences were noted between the results obtained from complementary test positions for mortar joints and brick face of uncoated panel assemblies.

As noted in the analysis of results for the modified E-514 tests, this variability may be attributed to localized material anomalies and minor imperfections introduced within the test panels during assembly. This phenomena was more pronounced for the water uptake tube tests given the highly confined nature of the test. At some of the test locations, the volume of the uptake tube was emptied instantaneously due to voids within mortar joints or through the seal between the uptake tube and the masonry surface. In these particular cases the results were discarded.

In comparison, the rate of water penetration obtained across masonry joints exceeded that through the brick face locations by a factor of 5 to 30 (approximately), disregarding high and low values. With regards to the reduction in the rate of water penetration across the panel assemblies at the mortar joints, reductions in the order of 30% to 100% in comparison to the uncoated or base panel assembly were obtained. Reductions in the order of 90% to 100% were obtained at the brick face test positions.

A reduction in the rate of water penetration in the order of 5% to 25% was also observed at the brick face for the control panel F. In comparison, an increase of approximately 50% to 60% was observed for the rate of water penetration in the control panel at mortar joints. This variation may be exposure related as climatic factors may have had an impact on the results of the experiments.

The limited data presented in this part of the discussion would indicate that water penetration test results obtained through the water uptake tube method are drastically influenced by the location at which the test is conducted. Given significant differences obtained between measurements at mortar joints and the face of the brick, it was clear that considerably more data would be required to accurately evaluate the performance of masonry coatings on a masonry substrate utilizing this method. As such, an evaluation of individual product performance cannot be undertaken. However, the available data indicates that as a whole, all of the products exhibited very high levels of water repellency.

With regards to changes in performance over time, four of seven water uptake tube tests conducted at mortar joints indicated a decrease in water repellency over time. In comparison, eight of twelve tests conducted at the brick faces likewise demonstrated a decrease in repellency over time. Given the limited data, no generalization can be made regarding the performance of repellents over an extended period, based on the water uptake tube method.

3.3 Air infiltration testing

The experimental data obtained prior to and following the application of the masonry coatings indicated no significant change in performance for any of the panel assemblies. As such, it was concluded that the application of masonry coatings had no impact on the air flow characteristics of the panel assemblies. In addition, very similar leakage characteristics were obtained for all panel assemblies (including coated assemblies) at various differential test pressures. Average leakage rates were determined to range from $0.23 \text{ l/s}\cdot\text{m}^2$ at 25 Pa differential pressure to $1.85 \text{ l/s}\cdot\text{m}^2$ at 300 Pa differential pressure. All air infiltration rate determinations were conducted prior to water tests.

4 Conclusions and recommendations

Three test methods were utilized over a period of seven months for the purpose of evaluating the change in performance of repellency induced by the application of coatings on six masonry panels. A summary of the results has indicated substantial improvements in the resistance to water penetration upon application of commercially available masonry repellents. This change in performance was primarily evaluated through the use of a RILEM water uptake tube method and by ASTM E-514 test

method adapted for the requirements of this particular study.

Although these improvements in water repellency were generally confirmed by these two test methods, limitations were noted for each of the methods. Several factors were identified which could influence or bias test results.

With regards to the modified ASTM E-514 test, slight, normally occurring imperfections in the wall assembly, including small openings in the mortar to brick interface provoked variations in performance. The importance of these small openings at the given test pressure of 500 Pa cannot be neglected as significant amounts of water may easily be transported through the presence of these small discontinuities through the masonry.

As such, the evaluation of the resistance to water penetration for a masonry panel (or any other assembly for that matter) with this method requires a larger sample population to better evaluate the effect of imperfections of the assembly on the test results. Otherwise, an evaluation based on the modified E-514 method would be more representative of a combined assembly performance (i.e. masonry & water repellent) rather than the performance of the repellent itself.

The water uptake tube method for the evaluation of water repellency also has limitations. By the very nature of this particular test method, results obtained by the water uptake tube method are representative for a highly localized area and are very susceptible to surface imperfections. This is especially important at points of material transition, as was confirmed for mortar joints in the brick panel assemblies. An accurate assessment of a material's water repellency or resistance to water penetration based on this method would require a significant number of tests distributed over a large surface.

Barring local material or assembly imperfections, the evaluation of a masonry coatings by the water uptake method should in theory be a better gauge for a comparative evaluation of coating performance (as opposed to that of the assembly). This is supported by the preliminary data in which all coating materials exhibited high water repellency characteristics, as evaluated for the brick face of the masonry. These same coatings, evaluated by the modified ASTM E-514 procedure, produced significantly different results.

The modified ASTM E-514 test data may have revealed a trend indicating an increase in the rate of water penetration over time in comparison to results obtained following the initial application of the masonry coating products. As mentioned earlier in this report, this phenomena may be due to product deterioration which would necessitate reapplication of some products after a given time interval. Additional investigation is required to evaluate this phenomena.

With regards to the air leakage characteristics of the wall assembly, the preliminary test data indicated no significant change due to application.

Although the resistance to water penetration is an important criteria in the selection of a water repellent, consideration to vapour diffusion performance is also an essential performance characteristic which must be considered prior to selection. The study of vapour diffusion through masonry panel assemblies coated with water repellents is currently being evaluated. Readers are cautioned in the interpretation of the test data without additional information pertaining to the vapour diffusion characteristics of the applied products and the durability of performance.

5 Acknowledgments

This research project was sponsored by Canada Mortgage and Housing Corporation (CMHC). Appreciation is expressed to the following individuals and organizations for participating in this project:

Mr. Rousseau (CMHC)
 Mr. Grenier (Institut national de la recherche scientifique)
 Mr. Patenaude (Air-Ins Inc.)
 Representatives of participating masonry coating Manufacturers
 Mr. Campanozzi (Restaurations DYC Inc.)
 Mr. Grignon (Briqueterie St-Laurent).

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54 EXPERIMENTAL PROGRAM TO EVALUATE BUILDING COMPONENTS SERVICE LIFE: FIRST RESULTS ON BRICKWORK

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Abstract

In this paper we report on preparatory work and results of an investigation performed in order to study experimentally the decay of the functional performances of some building elements by means of laboratory and outdoor aging. This experimental investigation is part of a larger on going research program agreed between DISET and LTS, whose goal is the development of a methodology for the prediction of building elements service life and for the modeling of degradation mechanisms. The aging tests have been performed on building elements belonging to the class of the external walls. The specimens are composed of perforated brickwork coated with render and of a protective layer consisting of an acrilical water painting. For comparison, the aging tests were also performed on a series of unprotected brickworks. The accelerated aging program, conducted within an environmental test chamber, develops on the iteration of a cycle consisting of four distinct phases in which are simulated the most significant climatic conditions and actions (rain, irradiance, freeze and thaw), to which the chosen building elements are generally exposed. The cycle has been calibrated so as to yield the right stressing load of the climatic agents on the basis of the time evolution of temperature and humidity measured during preliminary tests at different points inside the samples. Parallely to the laboratory aging program, outdoor aging will also be carried out. The decay of the functional performances of the specimens during the aging process has been recorded through the periodical measurement of some relevant material parameters by means of non-destructive and destructive testing methods. In a next and final phase, the experimentally collected data will be elaborated and interpreted in order to support and validate the proposed methodology for evaluating the service life of building elements

Keywords : service life prediction, durability, performance, brickwork.

Durability of Building Materials and Components 8 (Volume One). Edited by M.A. Lacasse and D.J. Vanier. Published in 1999 by NRC Research Press, Ottawa ON K1A 0R6, Canada. ISBN 0-660-17737-4

1 Introduction

The experimental program to evaluate the durability of building elements presented in this paper is part of a wider research context dealing with the behavior of elements over time. The DISET department has been dealing with this issue for a number of years.

The lack of experimental data about changing of the performances and functional characteristics of the elements due to the action of stressing agents, induced us to set up an evaluation method for durability to be applied to building elements. This method was presented during a national meeting held at Turin Polytechnic in October 1996 (Maggi P.N., 1996).

In this method, the durability is expressed in terms of propensity and can be deduced from the response of the material to the stressing agents and more precisely, from the decay of the characteristics related to the supposed performance (functional characteristics) which can be assumed by the analysis of the structure of elements.

The experimental program, conducted by LTS, and its first results presented in this paper aim at verifying the reliability of this method.

2 Experimental program

2.1 The building elements and the start-up of the experimental phase

The experimental phase will allow to evaluate the durability of some building elements out of a catalogue of external walls, whose typology is the most frequently used in Italian constructions. The building element chosen to start the experimental program is a traditional cavity wall composed by: internal hollow bricks, thermal insulating layer and external perforated bricks coated with render and finished with a water painting. The test will tackle only the external part of the wall, so the samples (39 x 51 x 13 cm) will be only composed by the external brickwork, the render and the protective coat. The specimen will have two configurations: protected with water painting and not.

The comparison between the results of the unprotected specimens and those of the protected ones will show the protective performance of the coating over time.

2.2 General testing layout

In order to evaluate the durability of building elements, it was chosen to submit the samples to laboratory and outdoor accelerated aging, as well as to outdoor natural aging. The choice to resorting to a double method of evaluation allows to correlate the different scales, thus avoiding to produce unreliable results or, at least, some that are not traceable back to outdoor environmental stresses. As far as the outdoor exposure is concerned, two sites with similar climate (Milan and Lugano) but with different levels of pollution were chosen. The samples will be placed southern oriented, vertically and with an inclination of 45°. This latter position allows the exposure of the building elements to conditions of greater stresses than the ones to which a vertical external wall is normally subjected. Thus three types of aging can be distinguished: laboratory accelerated aging, outdoor accelerated aging (45°) and outdoor natural aging (90°).

The outdoor accelerated and natural aging will start in April 1999.

2.3 Laboratory accelerated aging program

The first preliminary phase (May and June 1998) has consisted in verifying the stressing loads at different locations inside the sample through the continuous monitoring of temperatures and humidity. After this preliminary calibration phase of the cycle was ended, the laboratory accelerated aging program was started for the unprotected samples. To this purpose 14 samples were prepared.

At this date two samples have already concluded their stay inside the environmental test chamber: sample 10.1, with 7 weeks of aging test (148 cycles), and sample 8.1, with 13 weeks of aging test (285 cycles). The complete laboratory accelerated aging program will take about 8 more months.

3 The laboratory test cycle

In order to determine duration and structure of a complex test cycle able of reproducing the right environmental conditions, we first identified the outdoor agents affecting service life of the chosen building elements. Stressing agents were defined on the basis of both national and international standards (UNI and ICITE, ISO/TC 59/SC 3/ N 296, CEN, ASTM E 632, ASTM G 26, AFNOR - NF T 30-049).

The definitive aging cycle has a total duration of 6 hours and 25 minutes and is composed as follows:

1. *Rain.* In this first phase, whose duration is 60', the samples are sprayed with water at a temperature of about 20°C. The air inside the environmental test chamber is kept at a constant temperature of 20°C and at a relative humidity higher than 95% ; T_{12} =Transition time: 5' ;
2. *Freeze.* The wet samples are subsequently exposed to an ambient temperature of -20°C for 90' ; T_{23} =Transition time: 80' ;
3. *High temperature and high humidity conditions.* The ambient temperature is kept constant at 55°C and the relative air humidity is maintained above 95% for a duration of 60' ; T_{34} =Transition time: 5' ;
4. *High temperature and low humidity conditions with UV radiation.* During this phase the ambient temperature is lowered to 30°C and the relative air humidity is held at about 40%. The samples are irradiated by means of a Xenon lamp for a duration of 80'. The temperature at the surface of the samples is about 60°C ; T_{41} =Transition time: 5' ;

4 Characterization tests

The characteristics affecting the performances of the exterior walls under investigation were determined using the functional analysis method (Maggi P.N., 1998). This method allows to easily highlight the characteristics which influence the performances of the wall directly, the so called *functional characteristics*, and the characteristics which affect performances indirectly, the so called *related characteristics*.

In order to find out changes in functional or related characteristic, each specimen is analyzed before entering the environmental test chamber, as well as during and at the end of its stay inside the chamber. These characteristics are determined by means of both non-destructive and destructive tests, as shown in the Table 1.

Table 1: Characteristics measured on each specimen

Non-destructive tests	Destructive tests	Reference Standards
Weight measurement		
Weight loss measurement	Water vapor permeability	DIN 52615
	Water absorption	DIN 52617
	Porosity	SIA 162/1, test no. 5
	Elastic modulus	
	Bond strength	

On a weekly base, each specimen is photographed and weighted, and the amount of lost material is determined. The lost material is collected using fine meshes placed under each specimen.

Destructive tests are performed at the end of the planned exposure times of the specimens. The state of the render is characterized by the determination of its vapor permeability, water absorption, porosity and elastic modulus. The elastic modulus test will be determined according to the test method presented by Prim et al. (Prim, P., 1990).

5 Results

5.1 Non-destructive tests

The two photographs of Figure 1 show the same crack shortly after its appearance on the surface of one of the samples (left) and after 285 aging cycles (right). A deposit within the crack can clearly be observed. This deposit is probably due to soluble silica transported and released at the sample's surface during the dry phase of the aging cycle. The crack acts as a preferential path for water movement.



Fig. 1: Photograph showing a detail of the same sample after two different aging times.

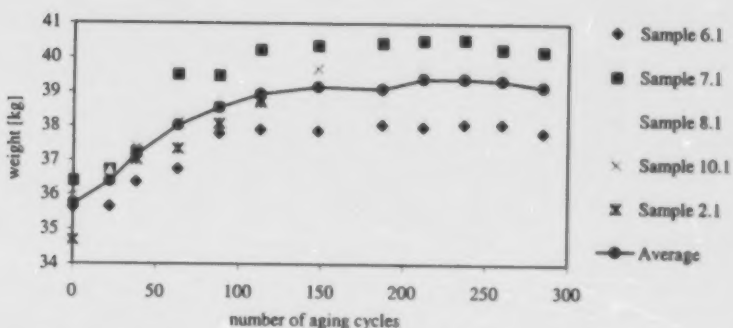


Fig. 2: Weight of the samples as a function of the numbers of aging cycles

In Figure 2 the weight of the samples is shown as a function of the number of aging cycles. There are two important facts to highlight: on the one hand each specimen absorbs a considerable amount of water during the test. In fact, the weight of the specimens increases from 7% to 12%. On the second hand, for all specimens the weight seems to reach its saturation value after about 150-200 aging cycles.

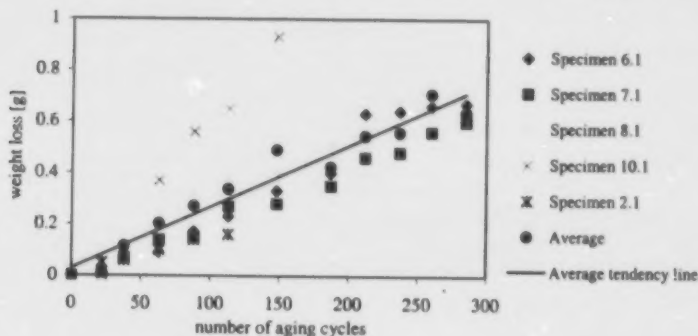


Fig. 3: Weight loss as a function of the number of aging cycles

The action of water, both in combination with freeze and thaw cycles and alone as washing away water, on the sample can be clearly seen on Figure 3. This figure shows the amount of mass loss due to the aging cycles as a function of the number of cycles. The tendency line computed on the average of the mass loss of each sample points out a linear trend.

5.2 Destructive tests

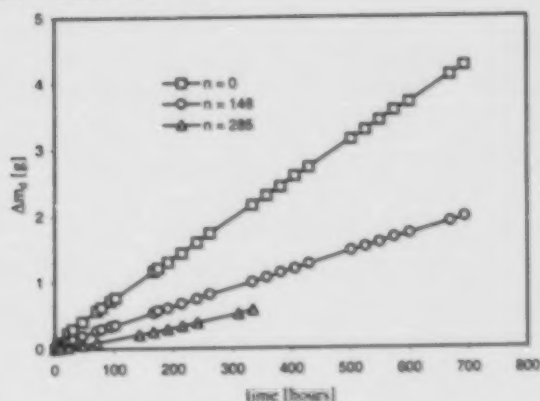


Fig. 4: Amount of water vapor which diffuses through the render as a function of time

In Figure 4, the amount of water vapor Δm_v , which diffuses through the render during the test of water vapor permeability conducted after exposure of the sample to $n = 0$, 148 and 285 aging cycles is plotted as a function of time. The test has been performed in accordance with the German standard DIN 52615 (1987) and at the following testing conditions: temperature 23°C, relative air humidity 0% vs. 50%. From the slope of the three linear curves plotted in the graph, it can be observed that the water vapor permeability of the render decreases for increasing number of aging cycles.

Table 2: Vapor diffusion resistance factor

	$n = 0$	$n = 148$	$n = 285$
μ	63,1	142,6	167,5
s_d (m)	0,8	2,2	2,6

In Table 2 the coefficient of water vapor diffusion μ and the thickness s_d of the diffusion-equivalent air layer, as derived from the curves plotted in Figure 2, are listed. The data show that the permeability of the render to water vapor decreases at least by a factor of two.

Figure 5 displays the amount of water Δm_a absorbed per capillarity by the render as a function of time after exposure of the sample to $n = 0$, 148 and 285 aging cycles. From the curves plotted in the figure it can be seen that both the rate of water absorption and the total amount of water absorbed until saturation decrease with increasing number of aging cycles. This behavior is also indicated quantitatively by the results listed in Table 3, where the values of the average absorption rate within the first six hours of testing (w_6) and of the total amount of water absorbed by the samples

(ΔW) are given. The test has been performed in accordance with the German standard DIN 52617 (1987) and pursued until total saturation by capillarity.

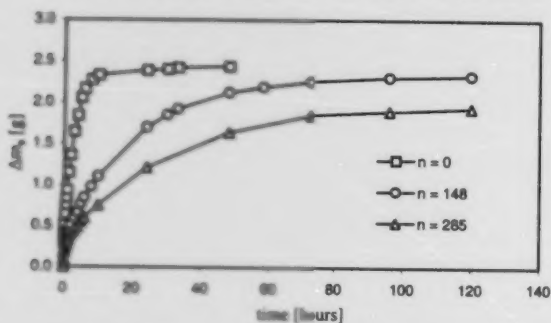


Fig. 5: Amount of water absorbed per capillarity as a function of time.

Table 3: Water absorption parameters.

	n = 0	n = 148	n = 285
w_s [$\text{kg/m}^2 \cdot \text{h}^{-0.5}$]	0.47	0.18	0.13
ΔW [g]	2.43	2.32	1.93

In Figure 6 the bond strength between render and brickwork is shown as a function of the number of aging cycles. The bond is measured in terms of the tensile stress required to detach from the underlying brickwork or cause failure of a core taken from the sample. From the data shown in Figure 6, it can be seen that the bond of the render increases for increasing number of aging cycles. After 285 cycles the bond overcomes the tensile strength of the render, as indicated by the different location of the broken surface (see Table 4).

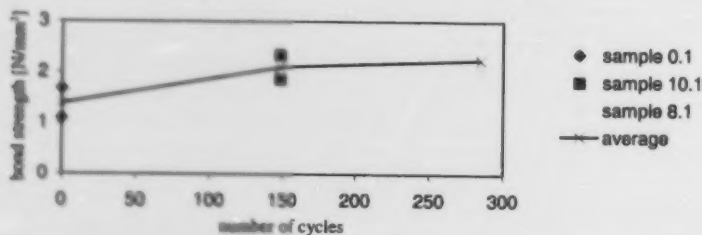


Fig. 6: Bond strength as a function of the number of aging cycles

Table 4: Bond strength

specimen id	nr. of aging cycles	σ [N/mm ²]	location of fracture	σ_b [N/mm ²]
0.1	0	1,10	render-brick interface	1,39
		1,68		
10.1	148	1,87	render-brick interface	2,10
		2,34		
8.1	285	2,60	render	2,25
		2,15		
		2,00		

5.3 Destructive tests on prisms

Prisms of render 4 cm wide, 4 cm high and 16 cm long were made when building the samples and were aged in the same environmental chamber where the samples were placed. The aging agents affecting the prisms are the ones affecting wall samples except for the UV radiation. Figure 7 shows a great increase of elastic modulus of the prisms over time (sample 15 and sample 16 are reference prisms).

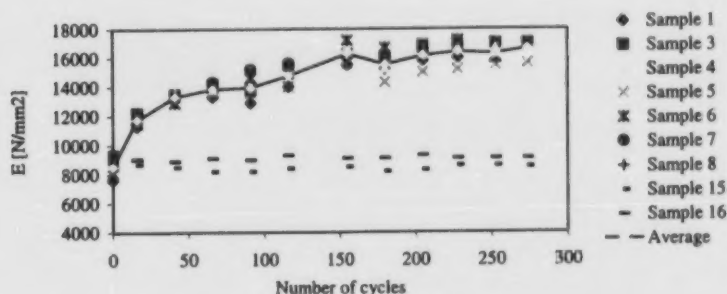


Fig. 7: Elastic modulus of the prisms

Table 5 lists the values of flexural and compressive strength of prisms of render as a function of the number of aging cycles. For comparison, the evolution with time of the same mechanical properties is reported also for prisms which have not been exposed to any aging cycle. It can be observed, on the one hand, that exposure of the prisms to the aging cycles results in an increment of both their flexural and their compressive strength, which become, respectively, two- and three-times larger. The strengths of the unexposed samples, on the other hand, remain unchanged, as it happens for the elastic modulus.

Table 5: Flexural and compressive strength

nr. aging cycles	age (days)	flexural strength (*) [N/mm ²]	compressive strength [N/mm ²]
0	7	1,9	7,3
0	7	1,8	7,5
0	28	1,8	7,6
0	28	1,8	6,8
0	120	2,0	7,4
0	120	2,0	7,6
148	105	2,5	18,6
148	105	2,4	17,9
285	120	4,5	21,4
285	120	4,8	21,1

(*) determined using simple beam with center-point loading

6 Future development of the research

The results obtained so far do not show a significant decay of the functional characteristics of the materials used. In fact, after 285 cycles, although in some samples little cracks have appeared, water vapor diffusion and water absorption by capillarity have decreased and the bond strength between render and brickwork has increased, thus indicating a reduction of permeability and an improvement of the mechanical properties. The same trend has been observed for flexural and compressive strength and modulus of elasticity measured on the prisms submitted to the accelerated aging. These results are in agreement with other unpublished data obtained at the LTS and also with results published by other authors (Bernsdorf et al., 1988, Künzel, 1994).

In the next months the aging tests on the unprotected samples will continue until a decay in the functional characteristics is obtained: for each characteristic it will be defined a critical level, under which the service life can be considered finished.

The first result we'll obtain is to estimate the experimental service life of the building component and of the constituting materials under accelerated aging testing.

A second result will consist in the comparison between the time behavior of the protected samples and the unprotected ones: this will give information about the degree of protection during time obtained by acrilical painting.

Outdoor aging testing on the same type of samples will be conducted in Italy and Switzerland to find out time re-scaling and this to obtain the real service life of the components for two different climates and pollution conditions.

The flapdoor of the aging chamber will be substituted by a section of the real building component and the aging cycles repeated to simulate its behavior between the outdoor environment (aging chamber) and indoor environment (laboratory at controlled conditions).

Once validated the experimental layout on this first reference component the aging program will be adopted for the building components of the catalogue of external walls, most used in Italy, to evaluate service life and to compare different products and components.

All this experimental data will be useful to support the original analytical material sensitivity evaluation method and to define a final durability and service life evaluation method.

7 Acknowledgments

Our thanks are due to the staff of the LTS, without whose assistance the results reported here would not have been possible, and to M. Melzi for its valuable cooperation in connection with the protective paint coatings.

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55 COMPARATIVE STUDY OF CLEANING TECHNIQUES APPLIED TO ANCIENT CONCRETE

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Abstract

Most of the buildings in urban areas are covered with dirt deposits, varying from thin black layers to black crusts. Historic buildings made of concrete, which for the oldest date back to the end of the 19th century in France, are not spared by such deposits which blur relief and colours. But these buildings that have to be preserved because they are the symbol of a specific architecture or because they are decorated with unique sculptures that are often very decayed. The aim of this study was to determine the most appropriate techniques to clean ancient buildings made of concrete and to preserve their skin. Ten cleaning techniques were tested and compared.

Keywords : cleaning, concrete, historic building, laser, poultice, sandblasting

1 Introduction

How to clean an ancient concrete which is both decayed and covered in black crusts? The objective of this study undertaken within the framework of the *Cercle des Partenaires du Patrimoine* (a French association based on a both financial and technical sponsorship between the French Ministry of Culture and several industries) was to assess the most suitable techniques to clean both ancient buildings and sculptures made of concrete without inducing any additional weathering. Accordingly ten cleaning techniques which are either usually employed for concrete or which are on the contrary derived from more original domains were compared. Each technique was tested in situ on the walls of the "Maison du Brésil" designed by Le Corbusier between 1957 and 1959 in Paris. The objective was to clean thin and

homogenous deposits of black dirt, preserving the wood-looking skin of the concrete. The performances were evaluated through an examination of the surfaces before and after cleaning and through the measurement of variations of colour and water permeability. The results reveal some heterogeneity in terms of both efficiency and aggressiveness of the ten cleaning techniques.

2 Ten cleaning techniques selected

The most frequently used on concrete cleaning techniques include probably pressure washing, sandblasting or the application of chemical agents (Virolleaud, 1990). But some of these techniques can be quite damaging (Philippon et al., 1992), especially when applied to ancient decayed concrete. For this reason, techniques using chemical cleaning agents were set aside in this study. Less damaging techniques currently used for stone cleaning in the field of historic building restoration (Ashurst, 1994; Andrew, 1994; Bromblet 1996), such as water spraying, poultices or laser cleaning, which all also seemed to be suitable for concrete too were also selected. Finally, two techniques not typically used on masonry or cementitious-based materials seemed to be of interest. The first one, originating from the cosmetic industry, is based on peeling. The second one, created for the removal of oil, paint or oxides on metals, polymers or glass, is an abrasive-based technique, using carbon ice particles. This last technique however was abandoned due to an eventual accelerated carbonation of the concrete.

2.1 Water-based techniques

Two water-based techniques were tested : pressure washing and water spraying. If pressure and water-flow have to be rather well controlled (Bromblet 1996) in the case of pressure washing, water spraying is simpler although it has to be completed with a hand brushing operation.

2.2 Abrasive projection

Cleaning techniques based on abrasive projection are the most numerous (Young et al., 1992). Their main differences come from the type of abrasive (sand, glass dust...), its shape (spherical, angular...), its size, and from the projection conditions (pressure, nozzle size...). Four techniques that mainly differ in the nature of their abrasives were chosen in this study (*Thomann-Hanry*, *Hydrofaçade*, and *Network* processes, and *Q*). These abrasives are respectively glass dust for the *Thomann-Hanry* process, spherical calcite particles for the *Hydrofaçade* process, angular calcite particles for the *Network* process, and angular alumina particles for the last one (*Q*). The *Network* process has a second specificity, which is its *Vortex projection* (Cf. fig.1), which is claimed to be less damaging on the substrate.

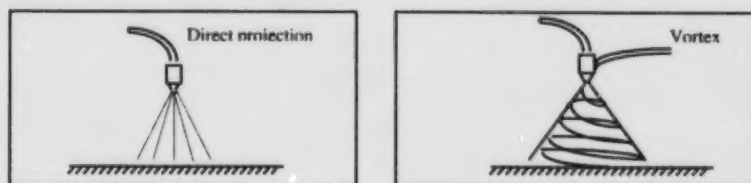


Fig. 1: Vortex projection of the *Network* process

2.3 Poultice-based cleaning

Two poultice-based techniques (the *Tollis* and *Lutum* processes) were also tested. These two treatments require more time than the six previous techniques. They consist in applying a poultice on the dirty surface in order to soften the black crusts. Thereafter the surface is softly brushed in order to remove the dirt.

- In the *Tollis* process, a network of micro-porous thin pipes is first fixed on the surface to be treated. Then a slightly wet poultice, made of stone-wool is projected onto this surface and packed down by hand. During the treatment, the poultice is continuously humidified using the network of pipes connected to an electronically piloted electromagnetic sluice gate.
- The *Lutum* process simply consists in projecting a specific wet poultice on the treated surface and letting it dry. When drying, the poultice becomes crumbly and easy to remove. In contrast to the stone-wool based one, this clay-based poultice is highly thixotropic and adheres to either vertical and horizontal surfaces.

2.4 Laser cleaning

Laser cleaning was specifically developed to eliminate dirt deposits on stone, but seems to be quite efficient on concrete too. Several studies on stone cleaning (Oriol et al., 1993) lead to the specific development of an ND YAG Q-switched laser, that emits Infra Red light (wave length : 10647nm) with a maximum energy of 400 mJ, and impulse times of 8 nanoseconds. The physical reactions involved with this laser generate a confined photo-removal. The laser impact creates a plasma, that sprays part of the dirt deposit, and produces a flash. A micro shock-wave is also generated, that for part of it spreads harmlessly in the substrate and for the other part, ejects the small particles of dirt that have not been eliminated with the plasma. The removal of the dirt layers can be gradual, which makes this cleaning technique quite soft, when the cleaning parameters are correctly controlled.

2.5 "Peeling" technique

The technique being evaluated (*K-Pack*) uses a Glycerol-polyvinyl-alcohol based product that can either be projected or applied with a paintbrush. It then requires 12 to 24 hours to polymerise. Once dry, it can be peeled off. If it becomes crumbly after polymerisation, it can be reinforced with a second layer, or even with a

cellulose fibre layer and another *K-Pack* layer. However the polymerisation of all the layers must be completed before the final peeling. The *K-Pack* is easily recyclable.

3 Testing conditions

The testing areas were walls, formed with alternatively vertical and horizontal wooden panels, protected from rain and covered with thin homogenous deposits of black dirt. An area of approximately 1.3m x 1.5m was cleaned using each technique (except for laser and alumina powder cleanings). The purpose was to eliminate the dirt deposit while preserving the surface and more specifically the wood-looking texture.

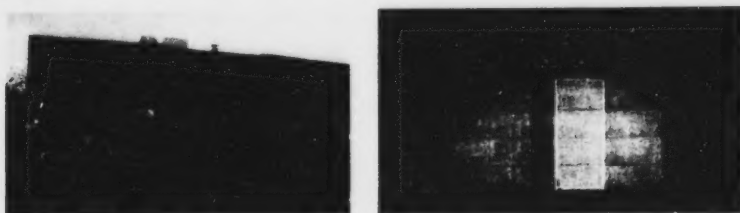


Fig. 2: *Maison du Brésil* in Paris, testing areas

For each technique, the cleaning operations were performed by specialists. The parameters and the duration of each of the ten treatments, which were quite different, are collected in Table 1.

Among the ten selected techniques, three that are not frequently used in concrete restoration and had to be adapted were tested with variations in their parameters : the *Lutum* process, the laser cleaning and the projection of alumina powders.

- The *Lutum* process was applied with poultices humidified either with water (*Lutum 0*) or with Diamine Tetra Acetic Ethylene (12.5g/kg of poultice) (*Lutum 25*).
- The laser employed was a BMI NL 101 (maximum energy - 400mJ; 80 cm focal length). Four areas measuring about 0.08m² were cleaned with several energy settings and laser to substrate distances, and with or without preliminary humidification.

- Dealing with alumina powder projection, several sizes of particles, pressures and nozzle/substrate distances were tested. As laser cleaning, this kind of technique which is more adapted to sculptures than to front walls lasts a little longer than the other abrasive-projection-based treatments. So the testing area was smaller (about 0.75m²) than for the other techniques (about 1.5m²).

Table 1: Parameters of the ten tested cleaning techniques

Technique	Type	Treatment Duration	Pressure / Flow	Nozzle size (mm)	d (nozzle/substrate) (cm)	Type of abrasive	Abrasive size (µm)
Pressure washing (PW)	water	a few min (1.5m ²)	200 bars	10	5-10	/	/
Water spraying (WS)	water	4h (1.5m ²)	/	/	/	/	/
Thomann-Hanry (TH)	abrasive	a few min (1.5m ²)	4kg	8	30-40	glass powder	50 to 100
Hydrofaçade (H)	abrasive	a few min (1.5m ²)	2 bars	10	30-40	calcite (spherical)	120 to 240
Network (N)	abrasive	a few min (1.5m ²)	2 bars	9	30	calcite (angular)	70% 60 to 80
Alumina (Q1)	abrasive	about 10 min (0.75m ²)	2 bars	2	10	alumina powder	45
Alumina (Q2)	abrasive	about 10 min (0.75m ²)	1 bar	2	15	alumina powder	29
Tollis (TO)	poultice	8 days (1.5m ²)	17 l/day	/	/	/	/
Lutum 0 (L0)	poultice	48h (1.5m ²)	/	/	/	/	/
Lutum 25 (L25)	poultice	48h (1.5m ²)	/	/	/	/	/
Laser (L1 to L4)	/	a few hours (0.08m ²)	/	/	15 to 80	/	/
K-Pack (K)	peeling	6 to 24h (1.5m ²)	/	/	/	/	/

Table 2: Specific laser cleaning parameters

Area	λ (nm)	Energy	Humidifying	Laser to substrate distance (cm)
L1	1064	maximum	yes	15
L2	1064	maximum	no	15
L3	1064	half	no	15
L4	1064	maximum	yes	80

4 Results

The aim of the study was first of all to evaluate the quality of each cleaning technique and also to determine its influence on concrete conservation. Hence the set of tests performed focused on three points :

- the evaluation of the efficiency of each treatment (visual observation, binocular, optic microscope and SEM, and colour measurements),
- the evaluation of the aggressiveness of the treatments (removed layers, poultice or abrasive residue in the porosity),
- the evaluation of the material and time necessary to clean a defined area (Cf. Table 1).

4.1 Visual observations

Simple visual observations reveal a great heterogeneity between the results of the ten treatments, some of them leaving part of the dirt, others producing a thorough cleaning resulting in a very white appearance.

- On the whole, the abrasive-projection-based techniques (TH, H, Q1, Q2 and N) lead to similar results, providing an apparently good cleaning efficiency. However it seems clear that a strict control of the parameters has to be carried out in order to avoid abrasion (particularly for techniques N, TH and H).
- Water spraying and the two poultice-based techniques (T0 and L25) also produced seemingly equal performances. The quality of the cleaning is however quite different from that of abrasive-based techniques, the walls looking whiter with water spraying and poultices.
- Dealing with laser, the performances were quite different on the four testing areas. The colour trends obtained vary from slightly grey (L3) to white (L4), sometimes with a yellow shade.
- Finally, three techniques (K, L0, PW) did not clean the testing areas sufficiently, some dirt deposit being still visible after treatment. In regards to the peeling technique (K1), a second and complementary application was made (K2). However this second application did not significantly improve the efficiency of the treatment.

4.2 Colour measurements

The colour measurements were carried out with a Chroma Meter Minolta CR110. Three parameters : "L" (increasing from black to white), "a" (increasing from green to red) and "b" (increasing from blue to yellow) were measured before and after cleaning (Cf. Fig. 3), but also on an area (NC) subjected to rainfall and that is quite "naturally" cleaned. The results match quite well with visual observations.

- The highest increases of the "L" parameter are produced using WS, T0, L25 and L4 methods. And on the contrary, the lowest variation of the "L" parameter were obtained with K1, K2 (K2 not being much better than K1), PW, L0, and N. An interesting point is that those last treatments lead to lower values of "L" than in the naturally cleaned area (NC).

- Dealing with the "a" parameter, none significant variation ($\Delta a_{\max} = 1$, for a parameter that can vary from -60 to 60) was measured with any cleaning technique.
- The evolutions of the "b" parameter are very heterogeneous. Some cleanings (L1, L2, L3, L4, and PW) clearly produced a yellowing (increase of the "b" parameter), where others generated a decrease (sometimes very low) of the "b" parameter (TH, TO, H, WS, L25, Q1 and Q2).

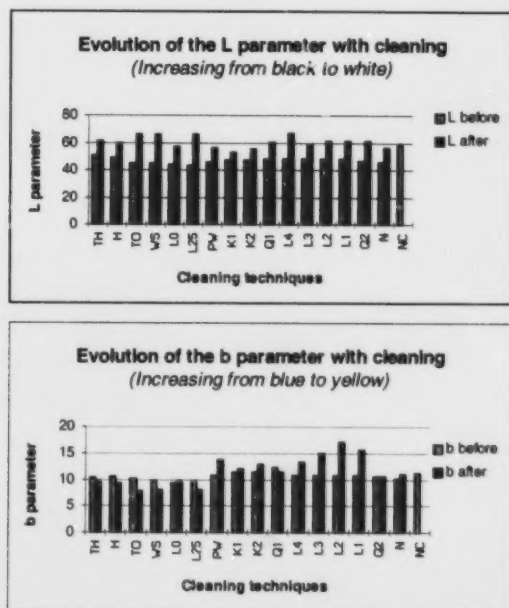


Fig. 3: Colour measurements : evolution of the "L", "a" and "b" parameters

4.3 Binocular examination

Cores were sampled on the cleaned/dirty limit areas for each technique. The surface of each sample was observed using a binocular. (Cf. Table 3).

- First, as expected, with PW LO, K1, and K2, some dirt particles are still observable on the treated surfaces, which confirms that those techniques are not efficient enough.
- But if the other techniques seemed to produce an efficient cleaning, they did not actually lead to equivalent results. With the abrasive-based techniques, as with the laser technique, a thin yellow layer or a yellow coloration appears under the

dirt deposit, that seems to be eliminated with the water based techniques (WS, TO, L25). This suggests that it is possible to dissolve this thin layer in water.

- In regards to the laser technique, the results differ from testing area to testing area. In the L4 conditions, the dirt deposits are eliminated, but the yellow layer (or coloration) as well. On the contrary, with L2 and L3 conditions, if the yellow layer (or coloration) is almost preserved (less with L2 than with L3), some dirt particles are still visible. The L1 conditions produced an intermediary result, with a complete removal of dirt, but with a partial disappearing of the yellow layer (or coloration).
- The examinations using a binocular also reveal that with some of the abrasive-based and with the two poultice-based techniques, some wastes of abrasive or poultice appear more or less in the porosity. The peeling technique also produced some product wastes shining on the surface.

Table 3: Binocular observations

Technique	Dirt wastes	Abrasive or Poultice waste
<i>Pressure washing (PW)</i>	+++	0
<i>Water spraying (WS)</i>	0	0
<i>Thomann-Hanry (TH)</i>	0 to ++	+
<i>Hydrofaçade (H)</i>	+	++
<i>Network (N)</i>	0 to +	++
<i>Q1</i>	0	++
<i>Q2</i>	0	++
<i>Tollis (TO)</i>	0	++
<i>Lutum 0 (L0)</i>	+++	+
<i>Lutum 25 (L25)</i>	0	0
<i>Laser 1 (L1)</i>	0	0
<i>Laser 2 (L2)</i>	++	0
<i>Laser 3 (L3)</i>	+	0
<i>Laser 4 (L4)</i>	0	0
<i>K-Pack (K)</i>	+++	+++
0 : none + : some ++ : a lot +++ : a lot more		

4.4 Optical microscope and SEM examinations

- Examinations of cores on SEM and thin sections using an optical microscope first reveal that the dirt deposit consists in a thin layer of gypsum (Cf. Fig. 4), containing several natures of aeolian deposits (calcareous or siliceous particles, fly ashes). It is also interesting to stress that this concrete soiled surface strongly resembles a dirty limestone surface.
- Those observations also indicate that the yellowing phenomenon is more likely due to the coloration of an existing layer (probably the gypsum layer) rather than due to an additional layer. Dealing with the origin and the composition of this

yellow coloration, several hypothesis can be expressed (coloration of the substrate by the dirt deposit, kind of patina...). A similar phenomenon exists with limestone cleaning and has been studied quite extensively (Vergès-Belmin et Al., 1993). However even though a number of interesting explanations have been offered none have yet been verified. Before analysing this yellow phenomenon more closely, it would first be important to verify that it is not specific to the concrete being examined and indeed, if it would occur with any other type of concrete.

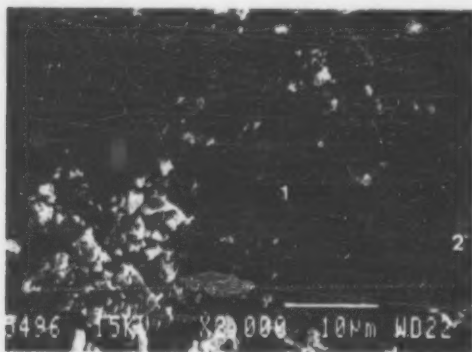


Fig. 4: SEM observation of the surface layer. 1 = Gypsum, 2 = Aeolian particle.
(Picture performed by LERM)

- In regards to both efficiency and aggressiveness of the techniques being examined, it is clear that some of them are not efficient enough (PW, K, L0 or L2) and that others are really too aggressive (L4 seems to turn the substrate into a nearly amorphous material, Cf. Fig. 5). For the other techniques, the observation of the microstructure does not introduce any additional information to the previous visual and binocular examinations. With any of those techniques, dirt wastes and gypsum concretions are partly eliminated. Except with the Tollis process which produces an almost complete removal of the gypsum layer and of the dirt particles. It would nevertheless be important to determine if it is necessary to eliminate this gypsum layer or if it has to be preserved.
- A last point to be made concerns the residues left by either the abrasive or the poultice techniques. The K-pack process, the poultices techniques (and more particularly the Tollis process, Cf. Fig. 6), and some of the abrasive-based techniques (Q1, Q2, N) produce quite a lot of residues. These residues are potentially propitious to biological developments (for example for K-pack residues) or could favour the fixing of new aeolian particles and accelerate the formation of a new dirt layer as a consequence.



Fig. 5: SEM observation of the surface layer. With L4 the substratum is getting nearly amorphous. (SEM picture performed by LERM)

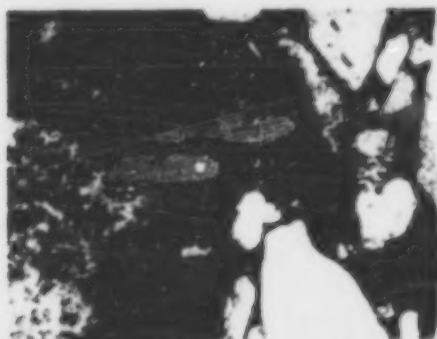


Fig. 6: Thin sections examined under optical microscope; residues of poultice with the Tollis process. (Picture performed by LERM)

4.5 Water permeability measurements

The water permeability was measured in-situ with Karsten-pipes, in accordance with 1978 RILEM recommendation n°II-4. This recommendation was designed for measurements on stone and was used here, as an extension, on concrete.

Unfortunately, it was nearly impossible to calculate any water permeability coefficient as the water-absorption was so low and no significant variation of the water permeability was revealed. This test is probably not appropriate for materials which are not very water permeable.

5 Conclusion

The aim of this study was to test some cleaning techniques in order to determine the most appropriate techniques to clean ancient concrete. Among the ten selected cleaning techniques, at least three do not seem suitable for the cleaning of concrete either because they are not efficient enough (PW, K, LO) or because they are too aggressive (L4). On the other hand, the other techniques produce different cleaning levels (e.g. elimination or preservation of the dirt deposits, gypsum layer, yellow coloration). Hence it is important to determine the most appropriate cleaning level in order to choose the most suitable technique. The testing areas will be closely examined over the coming years, in regards to new dirt deposits or biological developments, to evaluate the real impact of each cleaning technique. Regarding this last point, the examination of studies carried out in the field of limestone cleaning would be helpful since the surface layer of ancient concrete resembles that of limestone. Some techniques (poultice-based and abrasive-based) also produce several levels of wastes, that could have an impact on the durability of the cleaning operation (i.e. new biological developments). Once again, observing these test areas over the next few years is an important component in providing an over all assessment of these techniques. Regarding the abrasive-based techniques and laser cleaning, it is also important to note that cleaning conditions and user qualification are essential. Any of those technique can produce an abrasion of the concrete surface layer if the pressure or the energy is too high, if the nozzle or the laser beam is not at an appropriate distance from the surface or if the surface is not swept homogeneously. The duration of the cleaning operation can be one last important parameter and in the case of the ten tested techniques, this time varied from just a few minutes to 8 days. Finally, in order to confirm these observations and results, two complementary studies are scheduled in 1999. The same techniques will be compared at first on facades covered in thick black crusts, and then on sculptures made of concrete. The same testing protocol will be used except concerning the water permeability measurements that are clearly not appropriate and with surface roughness measurements that may be added.

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56 LONG TERM MIGRATION OF SILICONE INTO STONE AND ITS PREVENTION

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Abstract

A stone veneer finished curtain wall was developed in which stone was adhered and fixed to a steel curtain wall frame using silicone adhesive and metal fasteners. As the silicone adhesive was applied to the back surface of the stone, there was a possibility of contamination of the stone surface due to migration of silicone into the stone material, as this has occurred in past cases where silicone sealant has been used on stone surfaces. An investigation was carried out to predict the depth of silicone migration and determine the effectiveness of using a cover primer applied between adhesive and stone surfaces to inhibit migration. Chemical analysis was used to measure silicone migration within micro-size areas of the stone substrate. Based on this data, a theoretical study indicated that the migration could be analyzed as a diffusion phenomenon and that prediction of migration depth was possible. From the above results, the effectiveness of a cover primer was evident and the possibility of long-term prevention of contamination due to silicone adhesive was confirmed.

Keywords: Stone, silicone adhesive, curtain wall, contamination, chemical analysis, diffusion, numerical analysis

1 Introduction

A curtain wall system was developed using finished stone veneer which was adhered and fixed to a steel frame both by mechanical fasteners and elastic silicone adhesives (Sasaki and Nakayama 1995). Because the silicone adhesive was applied to the back of the stone, there exists a potential risk that the stone surfaces will stain

due to migration of silicone components from the back to the front fascia. As well, several cover primers were applied to the back of stone to investigate their effectiveness in preventing migration. Hence, being able to predict the likelihood of silicone migration on either primed or non-primed stone substrates was a primary objective of this study.

Past studies have attributed silicone related staining on building materials to the silicone component coming from the sealing materials (Ludwigshafen 1977; Nakayama and Takei 1988; Farmer et al. 1996), where the staining effect on stone or ceramic tiles was studied in either a quantitative or qualitative manner. However, numerical prediction of silicone migration from an elastic adhesive through stone and its preventive measures have never been studied with the exception of our previous study (Sasaki and Nakayama 1994). In these recent studies, the distribution of silicone in the stone specimens at a microscopic scale was determined by chemical analysis. From these results, numerical analysis of the silicone migration was completed thus providing a basis for developing a quantitative method to evaluate and predict the migration depth of silicone components in stone.

2 Specimens

2.1 Materials used

2.1.1 Stones

Chemical analysis was used to determine the amount of silicone migrating into the stone. When minerals in the stone contain silicon, the determination of silicon content due to the migration products must be compensated for by evaluating the background concentration of silicone in the stone. Many stones include varying quantities of silicate minerals such as quartz, feldspar and mica. Marble, however, is solely composed of calcite and contains virtually no silicone-based minerals. Hence, tests were conducted on marble stone from which the migration of silicone could more readily be determined. An Italian marble (Bianco Carrara) was selected for specimens of 50 x 50 mm size and 18 mm thickness (Fig. 1). This marble was composed of nearly uniform calcite particles of 0.2 to 0.5 mm in diameter and no other mineral was found as was confirmed by Laser Microprobe Analysis (LMA).

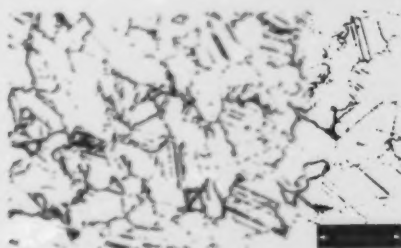


Fig. 1: Polarization microscopic image of Bianco Carrara

2.1.2 Elastic adhesives

The elastic silicone adhesive was two-pack, alcohol type. This adhesive was a high modulus product and had been used for the structural sealant glazing construction, a pane fixing method. Silicone sealant and silicone adhesive contain organopolysiloxane as a major component, calcite or inorganic pigments as secondary components, aminoxysilane as a hardener and no plasticizer. Therefore silicone was considered the only component that might transfer from the adhesive to the stone.

2.1.3 Primers

Primer is generally used to improve bonding strength between adhesives and adherents. However in this study, primers were used as a shielding material to control the migration of the silicone into stone. Four types of primers were used, three of which had a high shielding capability (i.e. shielding primer) and one that is typically used for conventional purposes (i.e. standard primer). Primer components and amounts of coating are shown in Table 1. The shielding primers were mainly composed of an epoxy resin containing no silicon whereas the standard primer was composed of a silane-based compound containing approximately 5 % (wt.) silicon.

2.2 Preparation of specimens

The shielding primers were coated on the back of the marble specimens. After curing for one day, the standard primer was re-coated. Control specimens were made without any shielding primers such that the experiment consisted of four types of specimens: A to D. When the drying of primers was confirmed by touch, spacing bars of 5 mm wide and 10 mm thick were set on the perimeter of the coated surface to form a 40 x 40 mm and 10 mm thick enclosure. The silicone adhesives were carefully placed within this enclosure in such a manner as to avoid any entrapped air. In accordance with the Japanese Industrial Standard JIS A 5758, pre-curing was executed for 7 days under the standard conditions followed by post-curing for 7 days at a temperature of $50 \pm 3^\circ\text{C}$. Dimension of the specimen are shown in Fig. 2.

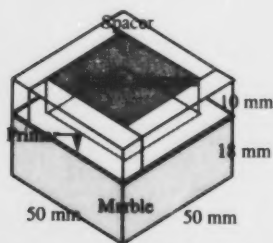


Table 1: Types of primer

Class	Component	Type	Spread (g/m ²)
A Standard	Silane	1 pack-solvent	100
B Shielding	Epoxy	2 pack-water sol	100
C Shielding	Epoxy	2 pack-solvent	100
D Shielding	Epoxy	2 pack-solvent	500

Fig. 2: Shape and dimensions of specimen

3 Method of silicon content determination

3.1 Procedure

The cured specimens were subjected to outdoor exposure test from April 20, 1992. In order to simulate real working conditions of silicone migration, specimens were set horizontally at a height of 1 m from the roof concrete surface with the marble surface up.

After 1.25 years, each one of 4 types of specimen were collected and subjected to quantitative analysis of silicon concentration in the marble after 48-hour conditioning under the standard condition. After 3.75 years, silicon concentration was determined in the same way and compared with that of numerical results based on the data of 1.25-year.

3.2 Silicon content

3.2.1 Electron probe microanalyzer analysis (EPMA)

A specimen with the standard primer was cut at the center and the fresh cross-section was observed by EPMA (Type: JEOL - Model; JXA-8621MX). Silicon distributions throughout the cross-section of the elastic adhesive, primer and stone were determined in weight percentage. The total distribution picture was determined by a cross-sectional analysis and subsequently a typical distribution was obtained by line analysis.

3.2.2 Laser microprobe emission analysis (LMA)

Specimens A to D were prepared in the same way as used for EPMA and silicon concentrations in the marble were determined in every 0.10 or 0.25 mm, a total 5 to 9 points in the thickness, by laser microprobe emission analysis (LMA). The crater diameter and depth resulting from the specimen evaporation due to laser emission were 100 μm and 40 to 50 μm respectively. Therefore the measured concentration represented an average silicon content in the crater volume. The LMA equipment was manufactured by Carl Zeiss Jena: LM10.

4 Results of silicon content determination

4.1 EPMA

After 1.25-year exposure, silicon concentrations in the elastic adhesive and in the marble near the interface of specimen A were 3 to 20 % and less than 1 % respectively. Considerable scatter of concentration in the elastic adhesive was observed, which may be attributed to the non-homogeneous mixing of two components in the elastic adhesive. The cross-section analysis provided a total picture of silicon distribution in an area, on which line analysis was subsequently executed. A typical result is shown in Fig. 3.

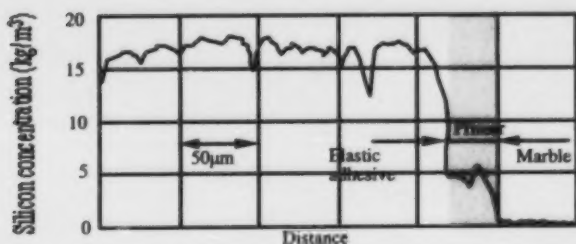


Fig. 3: Silicon distribution by EPMA plane analysis

The standard primer showed a thickness of approximately 30 μm where silicon concentration decreased sharply from approximately 5 % to less than 1 %.

4.2 LMA

Silicon distributions in each specimen after 1.25-year exposure are shown in Fig. 4. The silicon distributions, regardless of the type of the primer, showed the highest value at a depth of 0.10 mm from the interface and decreased inwardly. The extent of migration was as much as 1.3 mm for specimen A and 0.5 to 0.9 mm for the other specimens. It is evident that the shielding primers can prevent silicone components from migrating, especially for two-pack epoxy type primer D when coated to a level of 500 g/m^2 .

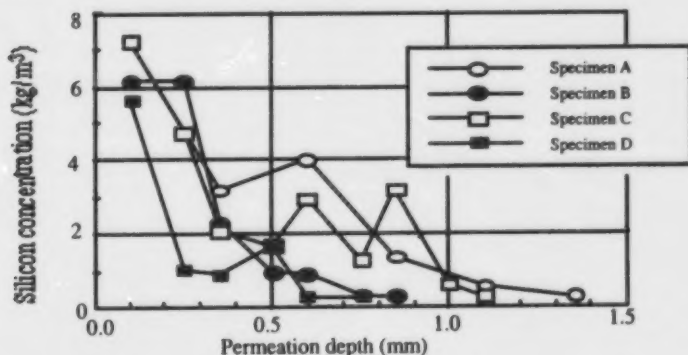


Fig. 4: Silicon distribution by LMA after 1.25-year exposure

5 Analysis of silicone diffusion

5.1 Basic assumptions

Because the silicone components that may migrate into the marble have no hydrophilic groups, interactions such as reaction, dissolution or convective transport between or with water in the marble can be neglected as well as interaction with calcite, a major component of marble. Therefore the migration of the silicone

component into the marble as was considered a pure diffusion process and flow would thus be driven by the concentration gradient.

In our previous paper (Sasaki and Nakayama 1994) the diffusion equation was solved assuming that a constant concentration of silicon was continuously supplied to the interface, an assumption based on the fact that only data for 1.25-year exposure was available. Having now additional data, derived from the 3.75-year exposure set, we reconsidered the boundary conditions of the diffusion equation and present a new numerical model.

5.2 Boundary conditions

We assumed following three boundary conditions.

1. Silicone component is not continuously supplied to the interface; the source emission is instantaneous. This means that silicone component is released only at an early stage and not supplied any longer as a result of polymerization. This assumption was derived from the time-dependent decrease of silicon concentration at 0.10 mm from the interface.
2. Initial silicon content in the marble is zero because marble contains almost no silicon, and the specimen is semi-infinite volume within a permissible error.
3. Diffusion coefficient is concentration-independent.

5.3 Solution

Balance equation of silicon under the above boundary conditions is given by:

$$\nabla(D \nabla C) = \frac{\partial C}{\partial t} \quad (1)$$

where infinite, plane and instantaneous emission source and concentration-independent diffusion coefficient are assumed. The marble microstructure, as previously shown in the micrograph, is nearly non-directional and can be treated as a one-dimensional problem. The one-dimensional diffusion equation yields:

$$\frac{\partial C}{\partial t} = D \frac{\partial^2 C}{\partial x^2} \quad (2)$$

Crank (Crank 1975) gave the solution to this equation as follows:

$$C(x, t) = \frac{M}{\sqrt{4\pi Dt}} \exp\left(-\frac{x^2}{4Dt}\right) \quad (3)$$

where C is the silicon concentration (kg/m^3), M is silicon emission intensity at primer/stone interface (kg/m^2), D is the diffusion coefficient (m^2/s), x is the permeation depth (m) and t is the time of exposure (s).

5.4 Determination of diffusion coefficient D and source emission intensity M

Using the data of 1.25-year exposure, we will determine the diffusion coefficient D and the source emission intensity M simultaneously and calculate silicon distribution after 3.75-year exposure. Thereafter, the numerical results will be compared with those actually measured. The data at 0.10 mm from the interface were neglected such as not to incorporate a portion of the silicon source itself. The natural logarithm of the equation (3) yields a straight line in the $\ln C$ - x^2 plane where the diffusion coefficient D , and source emission intensity M , were determined from the least-square fitted gradient and intercept respectively. An example for specimen A is

shown in Fig. 5 and results of determinations for all specimens are shown in Table 2.

Because the diffusion coefficient D , depends on temperature, the value obtained can be a diffusion coefficient at the average temperature during exposure. Different values for the diffusion coefficient D , were obtained in different specimens. This is likely due to the inhomogeneity of the material; slightly different values of D would be obtained depending where evaluations are taken.

5.5 Numerical results

Variations of silicon concentration in the time-space domain are as follows: for specimen A:

$$C(x,t) = \frac{3.1 \times 10^4}{\sqrt{t}} \exp\left(-\frac{6.4 \times 10^{13} x^2}{t}\right) \quad (4)$$

for specimen B:

$$C(x,t) = \frac{2.9 \times 10^4}{\sqrt{t}} \exp\left(-\frac{2.6 \times 10^{14} x^2}{t}\right) \quad (5)$$

for specimen C:

$$C(x,t) = \frac{2.6 \times 10^4}{\sqrt{t}} \exp\left(-\frac{1.1 \times 10^{14} x^2}{t}\right) \quad (6)$$

and, for specimen D:

$$C(x,t) = \frac{1.1 \times 10^4}{\sqrt{t}} \exp\left(-\frac{2.1 \times 10^{14} x^2}{t}\right) \quad (7)$$

Comparison of predicted and measured silicon distributions in marble at 1.25 and 3.75-year exposure are shown in Fig. 6. At 1.25-year exposure, predicted and measured values naturally agree well because the coefficients used for the prediction were obtained from these measured values. At 3.75-year exposure, the measured values of specimens B, C, and D with shielding primers showed smaller values than that predicted, though the result of specimen A showed good agreement. This may be attributed to changes in properties of the shielding primers or of specimens, due to long-term exposure. This later topic will be discussed later.

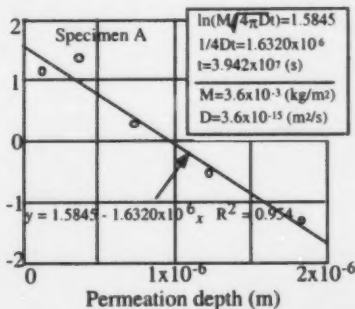


Fig. 5: Determination of diffusion coefficient D and emission intensity M from the LMA data

Table 2: Emission intensity M and diffusion coefficient D

Specimen	Emission intensity (kg/m^2)	Diffusion coefficient (m^2/s)
A	6.8×10^{-3}	3.9×10^{-15}
B	3.2×10^{-3}	9.7×10^{-16}
C	4.4×10^{-3}	2.3×10^{-15}
D	1.3×10^{-3}	1.2×10^{-15}

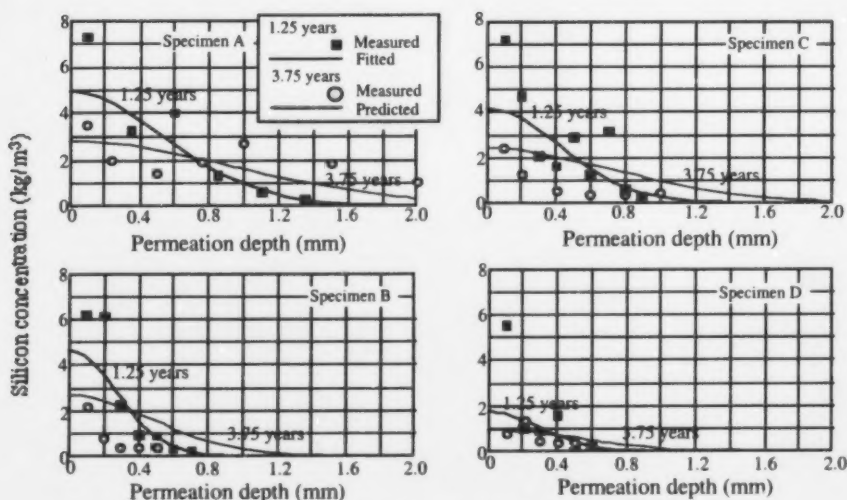


Fig. 6: Comparison of measured and predicted silicon distribution

However, this numerical model is successfully describing the overall time-dependent changes of silicon concentrations. Time-dependent behaviors of diffusion coefficient and source emission intensity must be studied and reflected to the numerical analysis in the future.

Prediction of long-term silicone migration after 20-year exposure is shown in Fig. 7 for specimen D with the most effective shielding primer and specimen A. The silicon permeation depth in specimen A was 6 mm after 20-year exposure whereas that of specimen D was 2 mm showing the preventive capability of the primer. In each case, within a normal service life the silicone component in the elastic adhesive can neither reach the marble surfaces nor cause stains if the marble is 30 mm thick.

5.6 Error analysis

5.6.1 Specimen thickness

A possible error factor in the numerical analysis is that the volume of the marble is not infinite (18 mm) unlike the assumption of semi-infinite marble thickness. Necessary marble thickness to reduce error less than the specified level depends on the amount of silicon in the marble, therefore the diffusion coefficient and exposure time are relevant. The amount of silicon existing between 0 and x can be given by:

$$\operatorname{erf}\left(\frac{x}{2\sqrt{Dt}}\right) \quad (8)$$

where it is known that the value of $x/\sqrt{Dt}$ should be more than 2.8 to reduce the error less than 0.01% (Fukai 1988). This results in a marble thickness of 13.9 mm at 50-years exposure capable of reducing error less than 0.01%, therefore the assumption of the semi-infinite volume used in the numerical analysis can be supported.

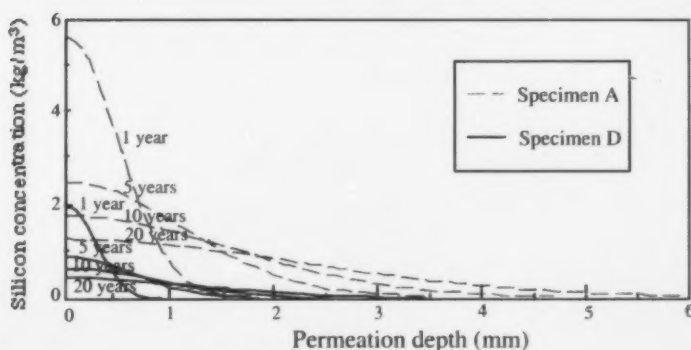


Fig. 7: Long-term prediction of silicone migration

5.6.2 Measurement error

The most significant measurement errors may be attributed to the LMA, where the microscopic portion of a specimen is heated by laser to a plasma and the emitted spectrum helps identify each element. The volume of evaporated specimen by which the emission intensity is calibrated, is determined by measuring the diameter and depth of the crater. The quantity of each element is then determined by the calibration curve. In this study, diameter and depth of craters were approximately 100 μm and 40 - 50 μm , therefore the spatial resolution of LMA was 100 μm . The first measurement, the point 0.10 mm from the interface, was likely to involve the primer itself, therefore this point was omitted from the determination of the diffusion coefficient D and source emission intensity M .

Accuracy of silicon concentration determination depends on the crater volume measurement whose accuracy is estimated to 3 % depending on the accuracy of measuring crater depth of 0.5 μm .

5.6.3 Effect of outdoor exposure

Specimens were subjected to outdoor exposure and not kept in the constant temperature-humidity chamber in order to evaluate the migration of silicone under conditions as realistic as possible. Therefore the measured values can represent the silicone migration when marbles are applied to the finish of curtain walls.

However, temperature-humidity variations and atmospheric SO_2 associated with the outdoor exposure may alter the microstructure of marbles (Gauri et al. 1989) resulting in the change of diffusion coefficient with time. These factors may be responsible for the difference between the numerical results and measured silicon concentration at 3.75-year exposure.

5.7 Summary of the results

We were able to model the migration of silicone into marble as a diffusion process with silicon concentration gradient as an apparent driving force. We assumed that the emission of silicon source was instantaneous, the silicone component in the silicone type elastic adhesive could transfer into marble only at an early stage and subsequently be fixed. Adopting this boundary condition to the diffusion equation, we

were able to calculate silicone migration profiles with time in a satisfactory precision.

With these results, we can conclude that the silicone component in the silicone type elastic adhesive begins to migrate into marble at uncured state immediately after construction but the migration may soon be terminated with the progress of polymerization of the adhesive which bound the silicone components.

Therefore, the importance of the shielding primers that can restrain the emission of an infinite amount of silicone component to a lower level was confirmed. We were likewise able to evaluate, quantitatively, the effectiveness of shielding primers by comparing the emission intensity M that can be estimated from the silicon distribution of 1.25-year exposure. According to this criteria, shielding primer D was the most effective.

6 Conclusions

Silicone component migration of silicone based elastic adhesives through marbles was quantitatively determined by EPMA and LMA. EPMA analysis has shown silicon distributions around the interface and overall silicone migration profiles from the elastic adhesive to the stone, whereas LMA analysis has shown microscopic silicon concentrations in the marble available for the subsequent numerical analysis.

Our numerical model assumed instantaneous source emission as the boundary condition and was able to evaluate the silicone migration over time. The extent of silicone migration into marble was numerically predicted. Evaluation of shielding primers applied to the back of marbles was also possible with this model and some shielding primers were proven to be feasible to prevent stains resulting from the migration of silicone component into stones. In order to execute more precise analysis, changes in diffusion coefficient due to outdoor exposure should be taken into account.

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57 THEORETICAL ANALYSIS ON THE THERMAL STRESSES OF CERAMIC TILE COATING SYSTEMS

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Abstract

The stresses on ceramic tile coating systems, subjected to temperature increase, were estimated by a 2-D finite element linear analysis. Physical and mechanical properties of the materials involved (ceramic tile, adhesive mortar, rendering mortar and grout) were determined experimentally and were used as model input. The numerical analysis has shown that peak stresses occur in the centre of the grout and decrease to nearly zero at the centre of the tile. The model is very sensitive to the tile Coefficient of Thermal Expansion and Young modulus and insensitive to the same parameters at the mortar layer.

Keywords: ceramic tile coating, finite element analysis, thermal stresses

1 Introduction

In spite of great development within the ceramic industry, problems related to ceramic tile detachment on buildings are very common at earlier ages or after a long period of adequate performance.

Since earlier ages, each layer of ceramic tile coating system (substrate, rendering, grout, adhesive mortar and ceramic tile) suffers differential volumetric changes due to factors inherent in the materials (drying shrinkage of mortar, moisture expansion of ceramic tile) or external factors (changes in temperature and/or humidity). Since free deformation of each layer is limited by the bond between them, stresses will likely occur.

Ceramic tiles are subject to temperature and humidity changes, especially on flat roofs and façades. The response of the materials to such variations appears in the form of differential volumetric changes between each layer of the system, causing stresses, which vary according to the magnitude of changes. As the properties of the materials are different, the stresses may be of important intensity. According to Billi et al. (1995), the coefficient of thermal expansion (CTE) of ceramic tiles varies from 6 to $7 \times 10^{-6}/^{\circ}\text{C}$. This value is two times lower than cement-based materials (CTE around $15 \times 10^{-6}/^{\circ}\text{C}$ according to Illston, 1994) in the form of rendering, adhesive and grout mortars. This difference can result in serious problems, especially in places and regions where great and rapid temperature changes may occur. Large falls of temperature may also produce stresses high enough to detach ceramic tiles after a long period of adequate performance.

It is possible to estimate the stresses on the various layers of a ceramic tile coating system subjected to temperature changes, by numerical modeling of it (Bowman and Banks, 1995). It is also possible to ascertain the influence of materials properties on the stress development.

In this paper, a 2-D finite element, linear elastic analysis was developed with the purpose to estimate the stresses. Experimental tests were performed to obtain data for the model, such as temperature variation through the system layers and physical and mechanical properties of the materials. Values obtained by other researchers were applied when there was no possibility to perform tests.

2 Experimental procedure

2.1 Materials characterization

Ceramic tile, adhesive mortar, rendering mortar and grout were studied to obtain values for model input. The ceramic tile used is called *gres*, obtained from powder pressing and two burning times, resulting in a high-performance material. Three adhesive mortars with different proportions of the polymer, methyl-hydroxyethyl-cellulose (MHEC), were studied. Table 1 shows the proportion mix of these mortars.

Table 1: Proportion of materials in adhesive mortars (weight)

	Cement	Sand	Filler	MHEC	Water/dry materials ratio **
RM*	1	2.8	0.2	-	0.18
M1	1	2.8	0.2	0.001	0.18
M2	1	2.8	0.2	0.020	0.18

* RM = reference mortar (without polymer)

** the weight of MHEC was not considered

Rendering mortar consisted of a mix of Portland cement, medium sand and hydrated lime, in proportions of 1:1:6 by volume (cement:lime:sand) and water/cement ratio of 1.76 (Mohamad, 1998). Grout mortar, used to fill the joints between each

ceramic tile, contained Portland cement, fine sand, and MHEC polymer. The proportions used were 1:3:0.002 (by weight).

The coefficients of thermal expansion (CTE) of the materials described above were determined, using Netzsch Model 202 and Shimadzu Model 50H Thermomechanical Analysers (TMA). Young's modulus and Poisson's ratio of the materials were also determined. The mortars were tested in compression according to NBR 8522/84 (Brazilian Standard for Young's modulus determination). The specimens of ceramic tile were cut in dimensions of 35 x 35 x 8 mm, and were also tested in compression. Table 2 shows the CTE, Young's modulus and Poisson's ratio obtained for each material.

Table 2: Average Coefficient of Thermal Expansion, Young's modulus and Poisson's ratio for mortars and ceramic tile

Material	CTE ($^{\circ}\text{C}^{-1}$)	Young's modulus (MPa)	Poisson's ratio
Ceramic tile	4.6×10^{-6}	38779	0.10*
Grout mortar	10.0×10^{-6}	12265	0.18
Adhesive mortar RM		22790	0.18
Adhesive mortar M1	10.0×10^{-6}	20739	0.18
Adhesive mortar M2		10745	0.18
Rendering mortar	9.6×10^{-6}	5918**	0.17**
Substrate	6.5×10^{-6}	20000	0.07
Mortar	9.6×10^{-6}	8000	0.17

* estimated

** after Mohamad, 1998

2.2 Temperature profile

In order to estimate the temperature variation in ceramic tile coating systems, a laboratory test was performed on a small system specimen, as shown in Figure 1. The ceramic tile glazing was heated from 23°C (ambient temperature) to 70°C by contact with a copper plate attached to a coil with flowing hot water. Thermocouples installed at layer interfaces monitored the varying temperature profile in the system for 4 hours.

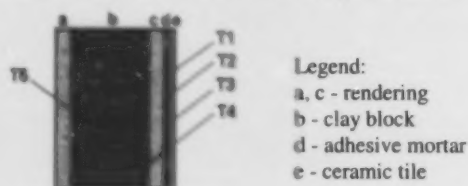


Fig. 1: Laboratory specimen and thermocouple positions.

Figure 2 shows the evolving thermal profile for the first 1.5 hours. These values were also used as input to the model.

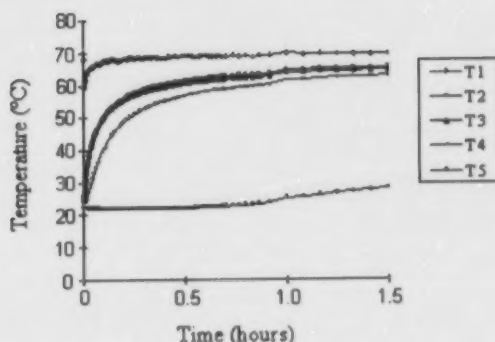


Fig. 2: Temperature profile in specimen.

3 Finite element model

3.1 Stresses due to thermal variation

To predict the thermal stresses on the rendering/adhesive mortar/ceramic tile interfaces, a 2-D finite element analysis was performed adopting a four-node plane element. The rectangular element shape suits the regular pattern of the tiling system. The model was considered isotropic and under plain strain.

The tiling system was modeled as a five-layer system (tile / adhesive / rendering / substrate / rendering) deforming linearly under a thermal loading. A full adhesive coverage was assumed. A panel, composed by 13 ceramic tiles (200-mm square) laid side by side on a ceramic block substrate, was analyzed. Taking advantage of the symmetry in the width direction, only half of the system was analyzed, with appropriate symmetry-boundary conditions. The boundary conditions adopted for this study were that the system was laterally fixed. The geometry used by the model is shown in Figure 3.

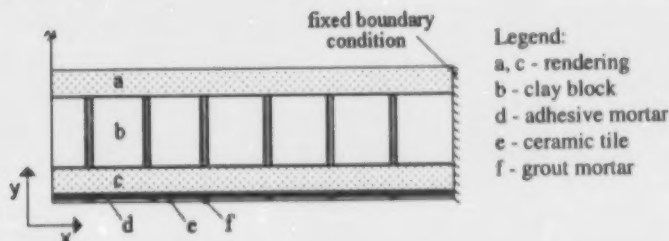


Fig 3: Geometry of the model.

The tile surface was heated at 70 °C. After approximately six minutes of heating, there was a temperature profile trough the system from 68 °C, on the tile surface, to 23 °C (the initial temperature) at the outer layer. As can be seen in Figure 2, this is the highest thermal gradient observed. Since it leads to the most critical situation, regarding stresses, the temperature profile (figure 2) obtained on this time was chosen to model the system. A linear temperature profile within each layer and a steady state were assumed.

To verify the model reliability, it was first considered homogeneous and isotropic. The tile mechanical properties presented on Table 2 were used to run this model. Since the system was free to expand in the y direction, the application of the thermal load did not produce stress in this direction. Therefore, a bi-axial state of stress was established, due to the boundary and the plain strain conditions. These stresses were expected to be equal and constant along an isothermal line. The stresses were compared to that obtained analytically by using the following equation (Timoshenko and Goodier, 1970):

$$\sigma_x = \sigma_z = \frac{E\alpha\Delta T}{(1-\nu)}$$

Where: - σ_x and σ_z are normal tensile stresses in the x and z direction respectively;

- E is Young's modulus;

- α is the Coefficient of Thermal Expansion;

- ΔT is the temperature differential;

- ν is Poisson's Ratio.

The heterogeneous system was then run to generate numerically the shear and tensile stresses. The material properties adopted to run the model are those from Table 2.

The shear and normal stresses (S_y) on the tile/adhesive mortar and adhesive mortar/rendering interfaces can be seen in Figure 4 and Figure 5. As can be seen, the stresses are nearly zero in the center of the tile and increase near the tile/grout interface. Close to the tile/grout interface, a region of concentrated stresses appears, whose signature depends very much on the material properties. At the tile/grout junction there is a change in the sign of the stresses which may be credited to the difference in the coefficient of thermal expansion of the materials. Young's modulus of the grout was three times smaller than that of the tile, allowing better deformation capacity for the grout. This movement differential may be considered as the main reason for the observed stress concentration. On the other hand, the adhesive is not thick enough to dissipate the stresses discontinuity observed at the tile/adhesive interface and, due to this factor, a discontinuity was also observed along the adhesive/rendering interface. Tensile stresses (negative sign) may cause the detachment of the tile.

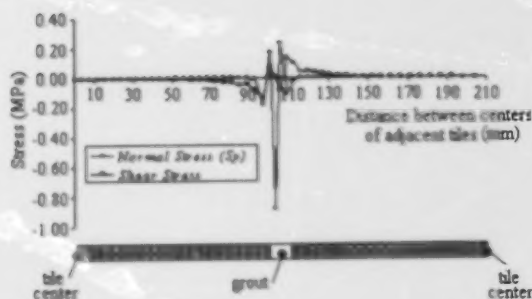


Fig. 4: Shear and normal stresses between centers of adjacent tiles, at the tile/adhesive mortar interface.

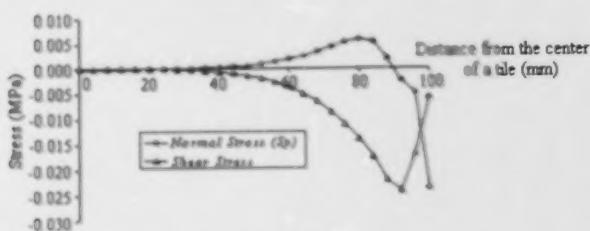


Fig. 5: Stresses of adhesive mortar/rendering interface.

3.2 Sensitivity Analysis

The sensitivity of the tiling coating system to the variation of Young's modulus and Thermal Expansion Coefficient of the tile and adhesive was investigated by calculating shear and normal (S_y) stresses. The variables were changed one at a time, according to the values shown in Table 3. In order to perform this analysis, only the interfaces of the tile located at the center of the system were analyzed. Three different values for the Young's modulus were chosen for the tile. The lowest and the highest values correspond to the lowest and highest adhesive stiffnesses found in the literature (Fiorito, 1994). The intermediate value was obtained from experimental testing. For the adhesive, four values were chosen: the lowest was obtained in the literature (Fiorito, 1994); the other three were taken from test results and correspond respectively to adhesives with highest, lowest and no polymers added to its formulae. Two different values of the thermal expansion coefficient were selected for the tile and adhesive mortar. Except for the lower value for the mortar adhesive, which was taken from the experimental programme, the other coefficients represent lower and upper limits from the literature (Fiorito, 1994; Billi et al., 1995; Illston, 1994).

Table 3: Mechanical properties of materials used in the sensitivity analysis.

	Tile	Adhesive
E (MPa)	30000	1050
		10745
	38779	20739
	70000	22790
α ($^{\circ}\text{C}^{-1}$)	4.6E-6	1.0E-5
	7.0E-6	1.5E-5

3.2.1 Effect of Young's modulus on the stresses

Figure 6 shows the influence of the tile stiffness on the development of stresses with temperature change. As can be observed, highest tensile stresses are likely to occur, in both the tile/adhesive and adhesive/rendering interfaces, by increasing the tile modulus. It was found that peak stresses rose by more than 100% by changing E by 79%. Shear stresses seem to be even more sensitive to the E variation (Figure 7). By changing E by 30%, peak stress increased by about 108%.

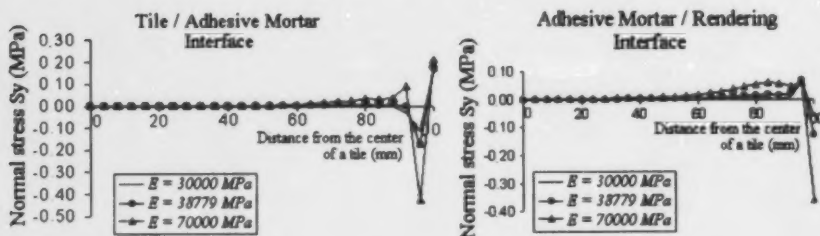
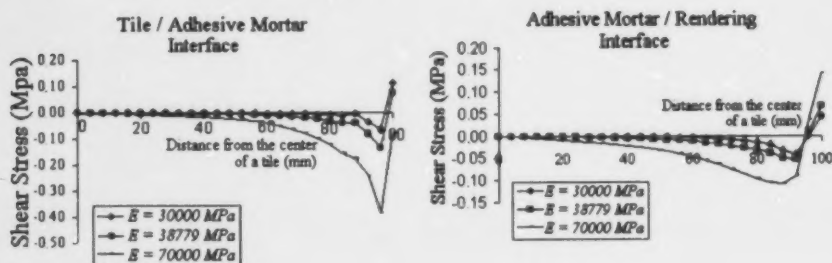
Fig. 6: Influence of the tile modulus on normal stresses (S_y).

Fig. 7: Influence of the tile modulus on shear stresses.

Regarding the influence of Young's modulus of the adhesive mortar, it could be noticed that the highest tensile stresses were obtained, in the tile/adhesive interface, for the most flexible adhesive (Figure 8 a). On the other hand, this value is responsible for the lowest stress profile in the adhesive/rendering interface (Figure 8 b). It is worth noticing that it was necessary to have a very huge change (about 900 %) in the E value to promote sensitive variations in the stress values. It might indicate that Young's modulus of the adhesive mortar is not important to the development of stresses.

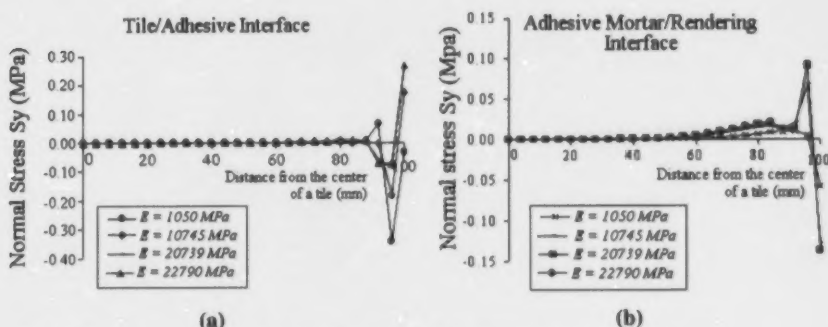


Fig. 8: Influence of adhesive mortar modulus on normal stresses (S_y).

Regarding shear stresses, it can be seen that the most flexible adhesive mortar does not produce high stress levels in the stress profile (Figure 9). Only with a very high increase in Young's modulus was it possible to obtain considerable variations in the critical stress values.

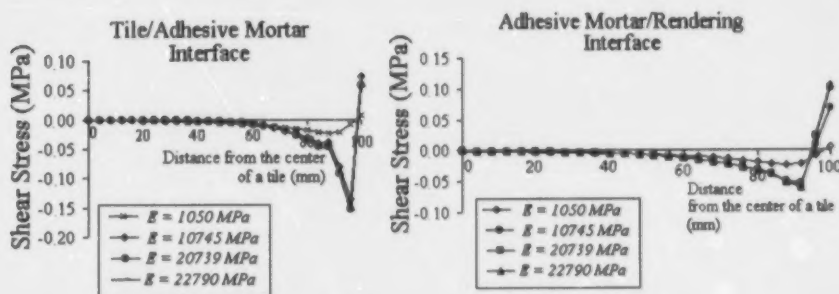


Fig. 9: Variation of shear stress with the adhesive mortar modulus.

3.2.2 Effect of Coefficient of Thermal Expansion on stresses

The sensitivity of the model to the Coefficient of Thermal Expansion of the tile was also tested. It can be seen in Figure 10a that, at the tile/adhesive interface, the S_y maximum tensile stress is slightly lower for the higher coefficient. For compressive

stresses, a higher variation is observed and, in this case, the higher coefficient produces a lower peak stress at the connection point with the grout. In the adhesive/rendering interface, the S_y stresses, compressive and tensile, increase with the Thermal Expansion Coefficient, as can be seen in Figure 10b. At the grout connection point, tensile stress is obtained. For both interfaces the higher the Coefficient of Thermal Expansion the higher the shear stresses (Figures 11 and 12).

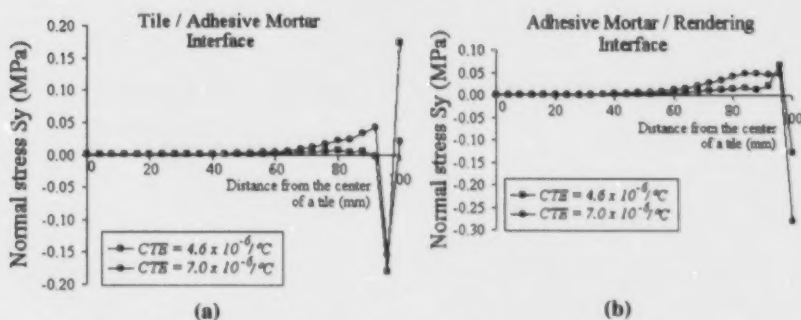


Fig. 10: Influence of tile Coefficient of Thermal Expansion on tensile stresses.

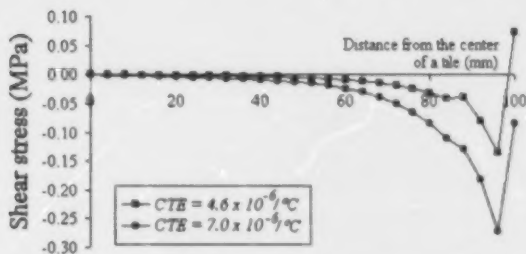


Fig. 11: Influence of tile CTE on shear stresses at Tile/Adhesive Mortar Interface.

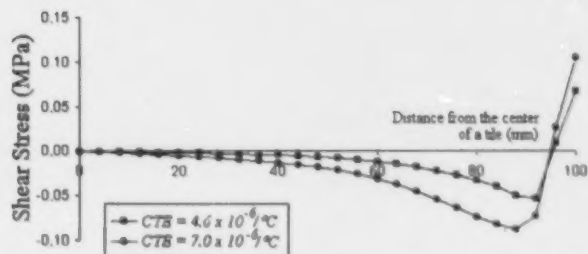


Fig. 12: Influence of tile CTE Expansion on shear stresses at Adhesive Mortar/Rendering Interface.

4 Conclusions

The model used to establish the stresses on a wall coated with ceramic tile has proved to be adequate. In all cases, stresses, shear and normal (S_y) are nearly zero in the center of the tiles, increasing near the tile / grout junction. This behavior might be attributed to differential movement of the materials due to their different mechanical properties.

The sensitivity analysis has shown that the ceramic tile coating system is more sensitive to the tile than to the adhesive mortar mechanical properties. Highest tensile and shear stresses were observed for stiffer tiles on both the tile / adhesive mortar and adhesive mortar / rendering interfaces.

The effect of the tile Coefficient of Thermal Expansion has proved to be smaller. Higher values of CTE produced lower peak stress at the connection point with the grout. With respect to the CTE of the adhesive mortar, the system is not as sensitive.

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58 MAPPING OF VISUAL DECAY FORMS AND INFRARED IMAGING OF STONE STRUCTURES FOR THE MAINTENANCE AND MONITORING STUDIES

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Abstract

Well-designed programs are necessary for the efficient maintenance and monitoring studies of stone structures. The mapping of visual decay forms and the infrared imaging of surfaces are useful methods to establish such programs for them. The studies have been performed on the 13th century caravanserai in Aksaray named Agzikarahan. The program started with site investigation by visual analysis of the decay forms on the stone. It involved the visual classification of the observed decay forms and their mapping on measured drawings and drawings produced from rectified photographs. The visual decay forms were classified in groups and each decay form was also evaluated as a degree of damage. The second stage was to take infrared images of the stone surfaces to produce temperature distribution maps which have further been analysed for moisture distribution in the structure. In the light of these studies, sampling has been done for the laboratory analysis. Juxtaposition of the different maps, photographs and infrared images have exhibited the present problems of the structure and their distribution and facilitated to make the immediate and long-term conservation programs.

Keywords: dampness, infrared imaging, maintenance, mapping, monitoring, stone structures, visual decay forms

1 Introduction

The frequency of the problems sourced from the improper or unnecessary maintenance and lack of maintenance show the importance of correct diagnosis of the problems for the maintenance of stone structures. In this respect the correct definition of the present problems of the stone structure and the detailed knowledge about its weathered state are necessary for the maintenance and monitoring studies. The

mapping of visual decay forms and surface temperature analyses of stone structures are the two non-destructive and useful survey methods to diagnose their problems which form the first stage of a well-designed research program. (BRE., 1981; Garrecht, 1996; Fitzner et al. 1997, 1996, 1995, 1994; Massari et al. 1993)

Two monuments have been studied for the definition and the diagnosis of the problems. One of them is Agzikarahan, a 13th century caravanserai in Aksaray, Turkey, and the other one is the tomb of Karacabey, a 14th century building in Ankara, Turkey. Agzikarahan has serious problems which can be detected visually, while the tomb of Karacabey has been restored in recent times.

The building stones of the Agzikarahan are tuff, used as cut stone and rubble stone. The characteristic wall structure is cut stone facing on both sides of the wall with rubble stone infill (Altintas). The observations and measurements were made on the tuff cut stone surfaces of the monument.

The site investigation has started with the mapping of visual decay forms as the first stage of the pilot study to prepare the rehabilitation programs for the related buildings. Detailed information were visually recorded on type, intensity and extent of the occurring damages as well as their distribution at the building. The second part of site investigation was the surface temperature analyses together with the infrared imaging of the building surfaces in the context of microclimatic investigations for searching the dampness in the structure. These studies were supported by material sampling and laboratory analysis.

2 Surveys, experiments and their results

2.1 The mapping of visual decay forms

The method developed by Fitzner and others was adopted for the mapping of visual decay forms of stone at Agzikarahan. (Fitzner et al. 1995, 1992) It involved the classification of the visually observed decay forms and their mapping on measured drawings and drawings produced from rectified photographs. The documentation of visual decay forms was done on 1/50 scaled elevation drawings. The decay forms were classified in three main groups such as "stone loss", "stone detachment" and "discoloration and deposits on stone surfaces". Each decay form observed at the monument is classified in the subgroups of these main groups. In addition, each decay form was also given a degree of damage such as "very severe decay", "severe decay", "medium decay" or "slight decay". The observed decay forms and their damage categories are as follows:

- Loss of stone material - most severe and severe decay:
 - Outbursts - most severe decay
 - Outbursts due to the anthropogenic influences - most severe decay
 - Alveolization - severe decay
- Detachment of stone material - medium decay:
 - Scales
 - Flakes

- Discoloration and deposits on stone surfaces - slight decay:
 - White staining
 - White-to-grey staining
 - Grey-to-dark grey staining
 - Black staining
 - Salt deposition
 - Biological colonization

The visual decay forms are analysed in three maps. The first map shows the loss and detachment of stone material and the second one shows the discoloration and deposits on stone surfaces. Furthermore, the areas of new repair stone and original stone are also recorded on the same drawings which is the third map to facilitate the comparison. For instance, Fig. 1 is the view of the north elevation at the courtyard and the analysis of decay forms observed on this view was given in Figures 2-4. During the survey period of seven months, the progress of stone decay forms have also been noted.

2.1.1 The results

The results obtained from the mapping of visual decay forms are summarized as follows:

- The most severe decay has been noticed on the original material at the lower parts of the monument up to 130 and 145 cm. height from the ground level as material loss: outburst, together with salt deposits. This situation indicated the existence of continual rising damp problem.
- Severe decay has been observed on the original material at the upper part of the monument as material loss: alveolization, together with lichens. This situation indicated a severe dampness problem at the upper structure which still continues.
- The distribution of new stone material use indicated the previous severe problems related with the upper structure and the lower part of the north elevation walls in the courtyard. Slight decay observed on these new stone surfaces as discoloration with white staining showed that the problems of the monument continue with the upper structure and the lower part of the north elevation walls in the courtyard. Furthermore, during the survey period of seven months, slight decay forms of discoloration in those areas have developed to medium decay forms as flakes and scales. It showed that monument needed urgent interventions.
- The maps also indicated the decay areas where representative sampling can be done for laboratory analysis.

2.2 Surface temperature measurements and infrared imaging of building surfaces

The study of the surface temperatures and the infrared imaging of the building surfaces is the part of the microclimatic investigation of the monument. Therefore, the temperature and humidity measurements were taken from the stone surfaces and in the atmosphere. The instruments used for the study are an infrared thermometer (Raytek Raynger PM Plus) and a thermovision camera (AGEMA Thermovision 550 camera) for surface temperature measurements, a Gann Hydromette B42 for the moisture content estimation in the material, and a Lambrecht Psychrometer Type 761 and Hanna HI 8564 thermo-hygrometer for the relative humidity and temperature measurements of the air next to the stone surface and in the atmosphere.

Although the surface temperature and humidity measurement have been taken for several periods during the four seasons, the infrared imaging of the building surfaces could be taken only once, in May. The analysis of the studies were done by the comparison of the surface temperature graphics as a function of height on the walls, the infrared images of building surfaces together with humidity measurements, furthermore, by the juxtaposition of the obtained results with the produced maps showing the distribution of visual decay forms.



Fig. 1: General view of the north elevation at the courtyard, Agzikarahan



Fig. 2: The map showing the areas of new repair stone and original stone

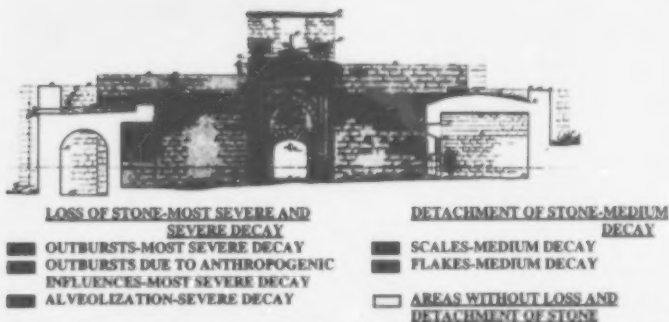


Fig. 3: The map showing the areas of loss and detachment of stone material

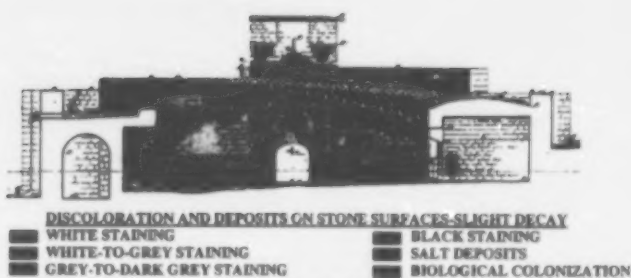


Fig. 4: The map showing the areas of discoloration and deposits on stone surfaces

2.2.1 The results

The analysis of these studies have shown that:

- The surface temperature graphics are consistent with the infrared images of the surfaces.
- There is a relationship between the surface temperatures of stones and their decay forms.
- The lower part of the exterior wall surfaces are the coldest areas of the structure. The upper parts of the exterior wall surfaces are the next cold areas. The photograph views and the infrared images of the lower parts and the upper parts of the structure are shown in Figures 5-8.



Fig. 5: Partial view of the south elevation at the courtyard



Fig. 6: Infrared image of the view shown in Fig. 5



Fig. 7: Partial view of the south and west elevation at the courtyard



Fig. 8: Infrared image of the view shown in Fig. 7

- The surface temperature increases with the increasing height at the lower parts of building according to the measurement taken in winter. The surface temperatures of the original stone with the most severe decay and severe decay are much colder than the surface temperatures of the new repair stones with slight decay (Fig.9). The moisture content in the original stone is very high, approximately 14%, and the same up to the height of 150 cm., according to humidity measurements taken in January. These results have indicated the presence of rising damp problem.
- In the following seasons the surfaces of the original stone at the lower parts of the building with most severe and severe decay are warmer than the surfaces of new repair stone. The extreme drying must have occurred at the surfaces of the decayed original stone in July and October.(Fig.9).

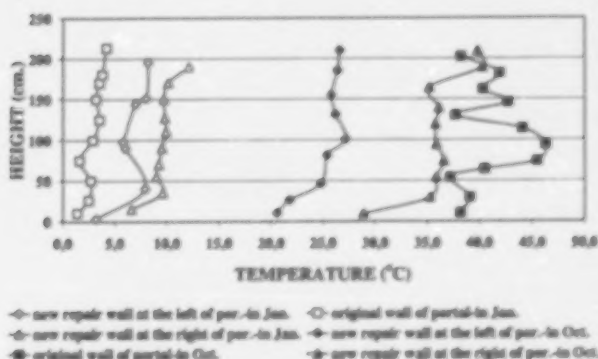


Fig. 9: The surface temperature curves for three region of the north elevation at the courtyard in January and October

- In the rising damp zone, the surface temperature curves of the original stone surfaces having the most severe and severe decays are inconsistent (Fig.10). Extensive salt deposits have been observed on those areas. One of the reasons of the surface temperature fluctuations should be the hygroscopic properties of soluble salts. In addition, the physical properties of the heterogeneously decayed stone should be checked.
- The surface temperature curves of the new repair stone with slight decay are more consistent in comparison to the original stones. For instance in Fig.11, the temperature curves have shown that the part of the wall up to approximately 35 cm. height is the coldest area, and the temperature is steady on the wall above this height up to 184 cm. The temperature starts to increase after this level where there is the passage from new repair stone to the original stone. Fig.12 is the view of this region and Fig.13 shows the infrared image of that area which is consistent with the temperature curves.



Fig. 10: The surface temperature curves of the original stone at the portal

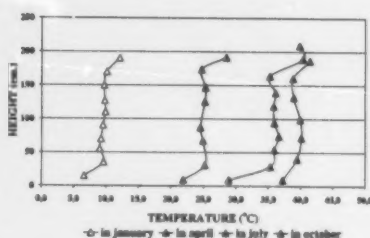


Fig. 11: The surface temperature curves of the new repair stone at the right of the portal



Fig. 12: Partial view of the lower parts from the north elevation at the courtyard



Fig. 13: Infrared image of the view shown in Fig. 12

- the surface condensation is unlikely on the exterior wall surfaces during the investigation times.
- The observations obtained from the infrared imaging studies are as follows:
- The infrared imaging gives the opportunity to analyse an area extensively while surface temperature measurements are restricted with location and time.
 - The infrared images have shown that different materials may have different surface temperatures for the same climatic conditions. In A* z* karahan, there are differences between the surface temperatures of new repair stone and original stone (Fig. 8 and Fig. 13). The infrared image of the tomb of Karacabey shows that the surface temperatures of mortar, brick and stone are different (Fig. 14).
 - The infrared images have given information about problem areas on the monument which can not yet be detected by visual analyses. For instance, the infrared image given in the Fig. 15, shows that a roof drainage fault has already started in the newly restored monument, the tomb of Karacabey.

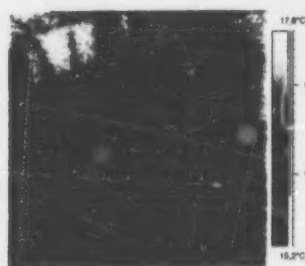


Fig. 14: The infrared image showing different surface temperatures for different materials

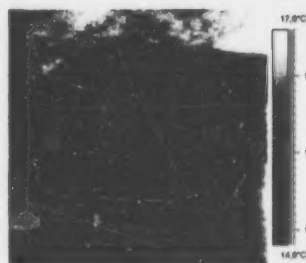


Fig. 15: The infrared image showing a roof drainage fault

- In the times and places where extreme drying occurs, the infrared images can be misleading for the dampness problems. The areas which have mostly suffered from dampness can be seen as dry surfaces in infrared images during a dry summer. Therefore a periodical study is necessary for the infrared imaging of the surfaces supported by temperature measurements.

2.3 Laboratory analysis

In the light of these studies, stone, mortar, salt and soil samples were collected from the decay areas. The laboratory analysis included both the analysis of physical properties of the materials, such as the density, the porosity, and as well as the qualitative and quantitative analysis of soluble salts. (Black 1965; Teutonico 1988) The laboratory analysis have been done to investigate the types of soluble salts, their sources and their relative amounts in the material to verify the importance of salts in the decay. The information obtained from the further laboratory analyses is also necessary for the assessment of the intervention techniques to the materials.

2.3.1 Results of laboratory analysis

The analysis has shown that the density and the porosity values of the original tuff samples are in the range of 1.57 gr./cm^3 - 1.70 gr./cm^3 , and 27% - 38%, respectively. The density and the porosity of the new repair stone samples are in the range of 1.56 gr./cm^3 - 1.67 gr./cm^3 , and 34% - 38%, respectively. Although there is not too much difference between the physical properties of original tuff and new repair stone, the new repair stone is slightly more porous than the original tuff stone.

The quantitative salt analysis have shown that the stone material contains considerable amount of soluble salts in the lower parts and the upper parts of the exterior walls, which are the damp zones.

The amounts of soluble salts in the original stone are in between 3.8%-9.2%. The original stone collected from the most severe and severe decay areas in the rising damp zone are very salty having the salt content of around 7%-8%. The average amount of soluble salts in new repair stone was found 6.0% at rising damp zone. On the other hand, the average amount of soluble salts was found to be 3.5% at the new repair

mortar. It is seen that the new repair stone contains more soluble salts than its neighbouring mortar.

The amounts of soluble salts in the new repair stone at roof parapet walls are in between 3%-8%.

The qualitative salt analysis have shown that the original tuff stone and the new repair stone at rising damp zone contain nitrate, nitrite, sulphate, phosphate and chloride ions in considerable amounts. These types of soluble salts sourced from the soil support the rising damp problem and the presence of ground water has to be checked. (Arnold et al. 1989; Nord et al. 1996; Schaffer 1972) In the new repair stone used at the roof parapet walls, sulphate, chloride, phosphate and nitrate ions have been found in large amounts. They may come from the deposited soil and plant on stone roof cladding. The presence of sulphate may be due to the use of cement mortar during previous repairs and the effect of air pollution should also be checked.

3 Conclusion

The mapping of visual decay forms and the surface temperature measurements together with the infrared imaging of the building surfaces revealed the types of problems and their distribution in the structure, as well as the extent of damages and their possible sources. The results also help to program the further necessary investigations. The obtained results are as follows:

- Juxtaposition of the produced maps and photographs have shown that the most severe decay was on the original stone at the lower parts of the structure which indicated the continual rising damp problem. The soluble salts were also important decay agents for the structure in damp zones and the presence of soluble salts in the stone supported the rising damp problem. The original field drainage systems have to be investigated in relation to the periodically produced maps of the structure and the substructure of the building together with the present topography.
- The upper parts of the walls are largely repaired with the new stones which is a sign of previous severe problems in those areas. The slight decay observed on these new repair stones has indicated the continuation of previous problems in the upper structure and roof drainage faults. The results have shown that the study of the original roof drainage system in terms of the materials used, its design principles, its defects, and its capacity is necessary.
- During the periodical mapping, the advancement or the slowing down of the decay can be followed, as well as the changes in the damp zones. The information are very helpful in monitoring studies.

Further non-destructive testing methods, such as ultra-sound velocity testing, are necessary to determine the present condition of the heavily deteriorated stone. The further detailed laboratory analysis are required for the removal of the soluble salts from the stone material, and for the selection of new repair stone and mortar.

These mapping studies supported by the laboratory analysis are the practical and non-destructive site investigation methods which are helpful at each stage of conservation, such as the planning of the immediate conservation program and the long-term monitoring and maintenance studies.

4 Acknowledgements

Thanks to the IZOCAM Company and its specialists Mr. Kemal Bayraktar and Mr. Seçkin Canoglu for the permission and technical assistance to use the thermal camera and its software; thanks to Mr. Burak Tavukçuoglu for the assistance during the studies at the site. Also thanks to the General Directorate of Pious Foundation for the permission to use the measured drawings of Agzikarahan monument

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59 COMPARING LABORATORY AND FIELD DURABILITY TESTING OF STONE

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Abstract

Research is being performed at Wiss, Janney, Elstner Associates, Inc. (WJE) laboratories toward developing an accelerated weathering test procedure that can be used to simulate the effects of natural weathering on stone cladding. Several stone types are currently undergoing laboratory and field testing at WJE. These tests are being performed to verify that an accelerated weathering test procedure used in laboratory studies can produce comparable results to the natural weathering of the same stone types. A non-destructive test procedure is used at set intervals during the weathering periods to determine the changes in the Relative Dynamic Young's Modulus of Elasticity (sonic modulus). The laboratory test specimens are usually monitored at 0, 10, 20 30, 50, and 100 freeze-thaw cycles while the field test specimens are being monitored semi-annually. The changes in sonic modulus using the accelerated weathering test procedure are similar to the changes determined from naturally weathered specimens. In addition, the changes in the sonic modulus have been shown to correlate to changes in the flexural strength property of the stone. Based on these tests, service life estimates can be determined for various stone types when they are used as cladding on buildings. The stones under evaluation for the current natural weathering studies are limestone, marble, and granite. We have accumulated three years of data and plan to extend the test program for a total of ten years. The results of this testing can be used as a guideline in developing clear specifications that address stone durability on an accelerated weathering basis.

Keywords: durability, stone, cladding, accelerated weathering, natural weathering, building cladding, laboratory testing, field testing.

1 Introduction

Durability is an important issue to consider when specifying stone for exterior exposure. In the past, it was only through experience that people could evaluate the durability of specific stone types. Experience is still a valuable tool, but it is now possible to perform tests that determine the life expectancy of a particular stone type that is to be provided for a specific building facade.

Stone is subjected to a variety of weathering conditions that are both natural and unnatural. Each type of weathering condition will produce different effects on the stone durability. Weathering is a result of changes in the body of the stone. Water and atmospheric gases are the reactants that cause the weathering process.

The rate of weathering is dependent on the macro and microenvironments. The macroenvironment is the ambient atmospheric weather conditions while the microenvironment is related to weather conditions at specific locations of a building facade.

1.1 Forms of weathering

The influences that affect the rate of weathering are temperature, moisture, and air pollutants. The average rainfall is a major controlling factor of the weathering rate because the rainwater will carry air pollutants into the stone by absorption. The absorbed rain and industrial pollutants will accelerate the weathering rate. It is these factors and the mineralogy that results in the variability of behavior for natural stone.

1.1.1 Chemical weathering

Stone exposed to air pollution may show signs of weathering by undesirable discoloring, loss of polish and surface erosion (delaminations and spalling). Air pollutants, or acid pollutants, can have serious detrimental effects on carbonate stones, such as limestone and marble. Rainfall that is absorbed into the stone is a controlling factor of weathering. The amount of CO_2 in the water causes the rate of dissolution of the carbonate minerals. Depending on the mineral composition of the stone, the water can cause the material to dissolve by percolating water solutions or by chemical decomposition. Higher temperature increases the chemical agitation of the minerals. Hence, such reactions are greater in tropical areas than in the colder temperature zones.

1.1.2 Temperature weathering

Depending on the location of a building, the temperatures on the surface of a dark stone facade can become as high as 88°C (190°F) during the summer and as cold as -23°C (-10°F) during the winter. The changes in temperatures can cause decomposition of a stone facade due to differential volume changes (expansion and contraction) of the various mineral grains that comprise the stone. The differing rates of thermal expansion of individual mineral types that comprise a single stone will cause the deterioration. All minerals expand with increasing temperature, but they expand at different rates and in different directions. The volumetric or linear expansion of the different minerals can cause micro cracking. The micro cracking will lower the strength of the stone and allow other weathering phenomena to enter and accelerate the disintegration process.

1.1.3 Freeze-thaw weathering

Damage to the stone can result as entrapped water expands under freezing conditions within the pores, cracks, and fissures that comprise the structure of the stone. The absorbed water forms expansive ice crystals that produce high compressive forces on the walls of the pores, cracks, and fissures. The compressive forces will produce tensile stresses on the crystalline bonds between the mineral components of the stone. Cyclic wetting, drying, freezing and thawing can eventually lead to fatigue of the crystalline bond and cause decomposition of the stone. Because of this, the tensile strength of the bond between mineral grain particles is of greater importance than the overall compressive strength when considering the durability of a stone.

The pores may also contain clays, some of which will expand when they become wet. The swelling of these clays will also produce large compressive forces within the stone. It may not be apparent that the swelling is occurring because the expansion of the clay may be a normal characteristic for the particular stone. The tensile strength of the bond between crystals may be great enough to resist the forces of the expansive clay. However, the stone may deteriorate rapidly when it is installed on the side of a building and the clay becomes wet and freezes.

2 Accelerated weathering testing

2.1 Test considerations

An accelerated weathering test procedure that can provide accurate durability information for natural stone must take into consideration the affects of temperature, water and atmospheric pollutants. Durability is an important issue to consider when specifying a stone that is to be used for exterior cladding on a building. Over the past forty years, an accelerated weathering test procedure for stone has been in the process of development. Results from the test have shown an excellent correlation to results from natural weathering.

2.2 Test procedure

The developed test procedure consists of placing a stone specimen with minimum dimensions of 30 mm (1¼ in.) thick by 100 mm (4 in.) wide by 380 mm (15 in.) long in a 4 pH sulfurous acid solution. The specimen is immersed only 6 mm (¼ in.) to 10 mm (3/8 in.) deep in the solution in addition to being set on 6 mm (¼ in.) diameter rollers in a stainless steel pan. The rollers assure that the entire stone face is subjected to the solution. The solution simulates acid rain on the exposed exterior face of a stone panel.

The specimen is then subjected to 100 cycles between a temperature range of -23°C and +77°C (-10°F and +170°F). A fresh solution is used after each 25-cycle interval. It is common to achieve four freeze-thaw cycles for each day of the testing.

Prior to the weathering test, the specimen is evaluated for determination of the Relative Dynamic Young's Modulus of Elasticity (sonic modulus) using procedures as outlined in ASTM C215, "Test Method for Fundamental Transverse, Longitudinal and Torsional Frequencies of Concrete Specimens." Sonic modulus testing is a non-

destructive means to evaluate the changes in the stone properties during the accelerated weathering test procedure. The test procedure entails spanning a specimen between two points and then tapping the specimen to produce vibration. A test device – the Rockland 5804A Analyzer as manufactured by Rockland Scientific Corporation – is used by WJE to monitor and record the frequency of the vibration. This test device is not the same as that noted in ASTM C215, but it provides identical test results. Depending on the durability of the stone tested, a change in frequency may be determined. The frequency and weight of the specimen are used as known variables in an equation to determine the sonic modulus. This provides an indication of the relative stiffness of the stone.

The sonic modulus testing is performed at a minimum of 25-cycle intervals. However, the most significant changes in sonic modulus are normally apparent during the first 50 freeze-thaw cycles of the testing. Therefore, it is usually beneficial to perform the testing at more frequent intervals during the first 50 cycles.

Unweathered companion specimens are tested for determination of flexural strength using procedures as outlined in ASTM C880, "Standard Test Method for Flexural Strength of Dimension Stone." In order to verify the changes in mechanical properties, flexural strength tests are also performed on the accelerated weathered specimens after the weathering.

3 Correlation studies between accelerated and natural weathering

Tests have been performed to confirm a correlation between data obtained from accelerated and naturally weathered stone. Results of these studies are provided in Sections 3.1.2 and 3.1.3. The studies were performed by subjecting similar stone types – both weathered and unweathered – to sonic modulus and flexural strength testing. The authors are not aware of others having performed such studies.

In several cases, stone specimens were obtained from attic-stock panels that had been stored within a building and, therefore, never subjected to natural weathering. Stone specimens were cut from the attic-stock panels and the exterior facade panels for the comparison studies. Charts were plotted using sonic modulus and/or flexural strength test data from both the weathered and unweathered stone. The charts were analyzed to determine the location of the tangent between the curve for the weathered stone to the curve for the unweathered stone that was subjected to the accelerated weathering test. The tangent provided an indication of the equivalent number of years of natural weathering to the number of freeze-thaw cycles of the accelerated weathering test procedure.

3.1 Marble studies

The accelerated weathering test procedure has been performed on many different types and groups of stone specimens for past projects. The following comprises a few examples of testing projects using marble that established a correlation between natural and accelerated weathering of stone.

3.1.1 Rooftop studies in Chicago, Illinois

About forty years ago, twelve domestic marbles were placed on the roof of a building in Chicago, Illinois. Sonic modulus testing was performed quarterly on the marble specimens over an eight-year period. The results of this testing are provided in Fig. 1. As can be seen, most of the marbles exhibited a leveling of the curves after 1½ years of natural weathering. Similar leveling of the sonic modulus curves has typically been evident after 50 freeze-thaw cycles of the accelerated weathering tests.

One of the stones used in the rooftop studies was Danby marble from Vermont. Recently, we had an opportunity to perform accelerated weathering tests using Danby marble. The accelerated weathering was performed for 100 freeze-thaw cycles. Figure 2 compares the results of the sonic modulus testing for the recently tested specimens with the previously studied rooftop specimens. A close correlation between the data is evident.

Based on the results of the Danby marble specimens, it was determined that approximately 100 freeze-thaw cycles of the accelerated weathering test procedure is equal to between 6 and 8 years of natural weathering in Chicago. Therefore, 12 to 16 freeze-thaw cycles can be considered equivalent to one year of natural weathering of a marble facade in a northern temperate environment.

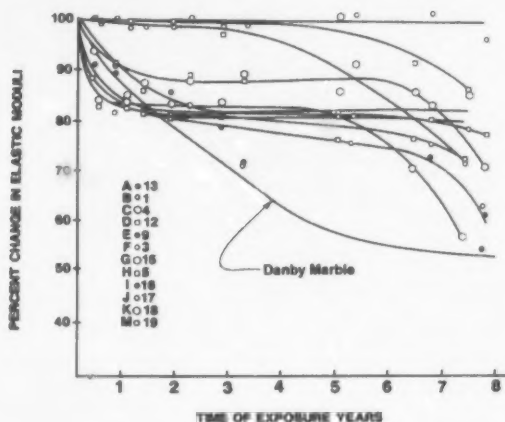


Fig. 1: Sonic moduli determined from rooftop natural weathering studies of twelve different marbles in Chicago, Illinois

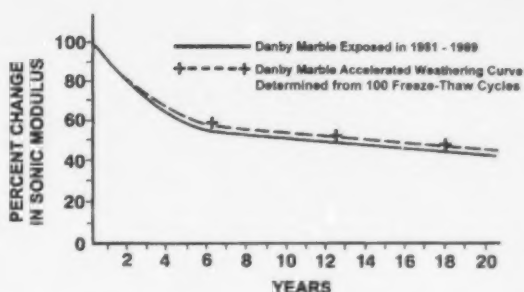


Fig. 2: Comparison of sonic modulus test results between the previous natural weathering studies and the recent accelerated weathering test

3.1.2 Marble-clad building in Chicago, Illinois

Studies were performed using flexural strength testing. Specimens were cut from panels of a building in Chicago that had been exposed to natural weathering for approximately 15 years. In addition, specimens were also cut from panels that had been stored indoors as attic-stock. The specimens were subjected to flexural strength testing at 0, 100, 200 and 300 freeze-thaw cycles of the accelerated weathering. The result of the testing for both groups of specimens is presented in Fig. 3. It can be seen that the charted curve for flexural strength is very similar to the charted curves from sonic modulus test data. This testing program further confirmed that about 15 freeze-thaw cycles of the accelerated weathering test equaled approximately one year of natural weathering for a marble facade in a northern temperate environment.

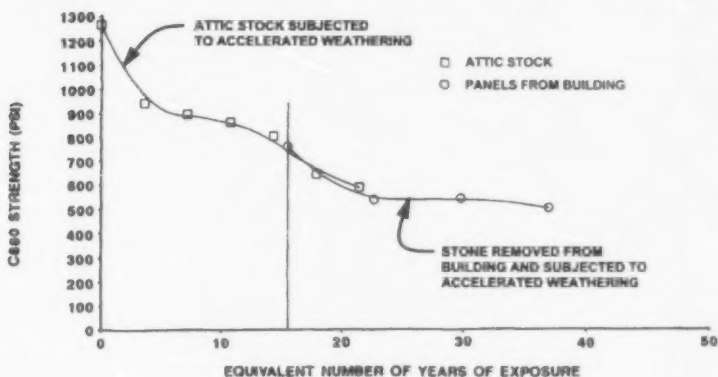


Fig. 3: Flexural strength versus years of natural weathering determined from marble clad building in Chicago, Illinois

3.1.3 Marble-clad building in Rochester, New York

Correlation studies were performed using natural weathered and unweathered marble from a 10-year old high-rise office building in Rochester, New York. Many panels from the building facade and panels that had been retained as attic-stock were provided for the accelerated weathering test. The results of the testing are summarized in Fig. 4. The chart shows a correlation between the sonic modulus and the flexural strength.

A weathering chart was plotted using the sonic modulus and flexural strength test data. The tangent between the sonic modulus test results of the weathered and unweathered data occurred at approximately 160 freeze-thaw cycles. Using these relationships, it was concluded that 160 freeze-thaw cycles of the accelerated weathering test procedure was equivalent to ten years of natural weathering on the building. Thus, 16 cycles were determined to be equivalent to one year of service life in upper New York State.

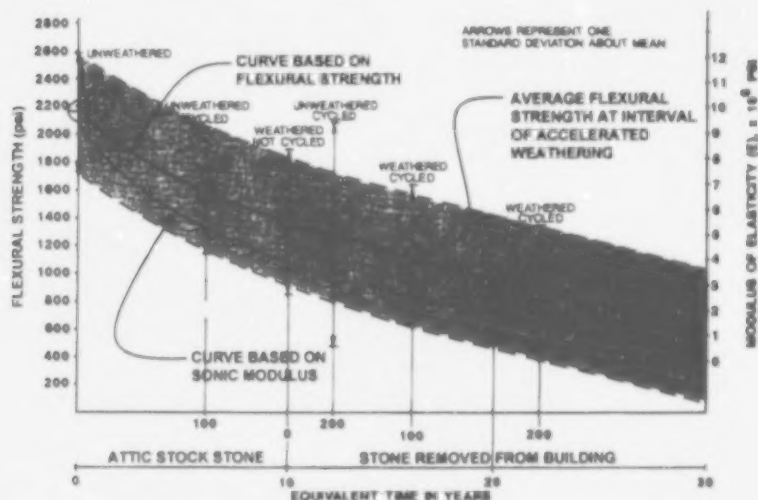


Fig. 4: Relation of number of accelerated weathering cycles to years of natural weathering for marble clad building in Rochester, New York

3.2 Granite and Limestone Studies

Similar data has also been obtained from granite and limestone specimens using weathered and unweathered stone. Changes in properties were plotted for 300 freeze-thaw cycles for granite, 500 freeze-thaw cycles for marble, and 200 freeze-thaw cycles for limestone. Based on the data, approximately 13 freeze-thaw cycles was determined to represent one year of natural weathering for granite, 12½ freeze-thaw cycles represented one year for marble and 12 freeze-thaw cycles represented one

year for limestone.

4 Current natural weathering studies

Over the past three years, WJE has been exposing granite, marble and limestone to natural weathering on the roof of a building in Northbrook, Illinois. The stone used for the testing had never been exposed to weathering. Five specimens are used for each stone type to provide a good statistical population for the studies. The specimens face south with a slight angle. Testing for sonic modulus was performed prior to the start of the natural weathering, as well as at six-month intervals. Testing and observations of these specimens is to continue for ten years.

Figures 5 through 7 are charts that show the current status of the sonic modulus test results after three years of natural weathering. Table 1 provides a summary of the approximate reductions in sonic modulus that have been obtained.

The natural weathering curves of the current rooftop studies can be compared with the previously shown charts that were prepared from the accelerated weathering test results. A similarity is discernable between the curves.

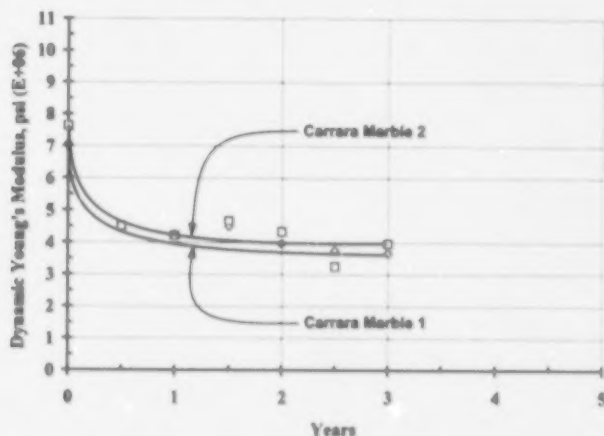


Fig. 5: Sonic modulus curves established for the marble specimens

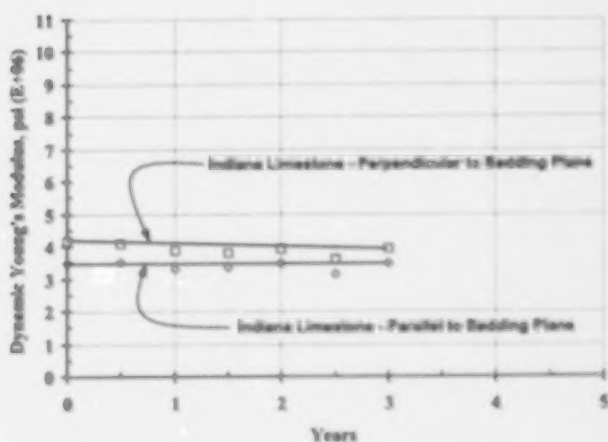


Fig. 6: Sonic modulus curves established from the limestone specimens

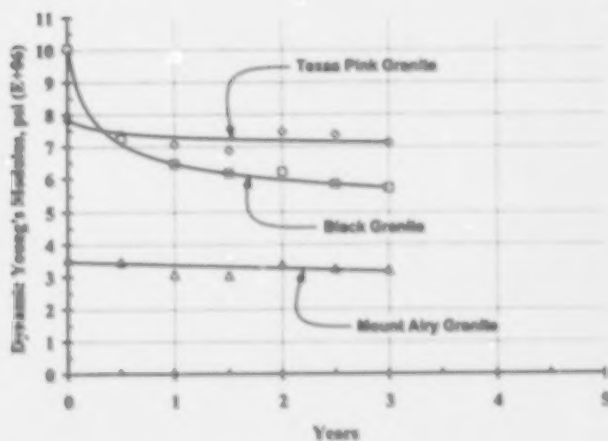


Fig. 7: Sonic modulus curves obtained from the granite specimens

Table 1: Loss of sonic modulus after three years of natural weathering

Stone Type	Approximate Loss (%)
Carrara Marble 1	47.9
Carrara Marble 2	48.2
Indiana Limestone - Parallel to Bedding Plane	0.0
Indiana Limestone - Perpendicular to Bedding Plane	6.5
Texas Pink Granite	8.7
Black Granite	43.1
Mount Airy Granite	9.6

The most significant losses in sonic modulus are apparent with the Carrara marble. In our past testing, we have often determined such loss in sonic modulus and flexural strength for various marble types.

The Black granite specimens are also showing a significant loss in sonic modulus. Most types of granite we have tested have performed well throughout the accelerated weathering test procedure, as is evident from the lower losses for the Texas Pink and Mount Airy granites. However, certain types of granite will exhibit larger losses in sonic modulus if they are composed of minerals that have weak crystalline bonds.

The slight loss in sonic modulus determined for the Indiana limestone specimens is common for this stone type. However, we have experienced instances where Indiana limestone has decomposed rapidly during the accelerated weathering test procedure. It was determined the deterioration of these specimens had resulted from clay inclusions in bedding planes of the stone.

Visual deterioration of the specimens is also recorded at each six-month interval. Besides the normal erosion that can be expected on the surfaces of the carbonate stone, only one Texas Pink granite specimen has shown deterioration with spalling at a corner.

5 Conclusions

Several years of comparison studies using accelerated and natural weathering has shown a good correlation between the test data. We have shown that the accelerated weathering test method can be used to evaluate a particular stone for durability. If naturally weathered and unweathered stone is available, sonic modulus test data can be used to determine a relationship between the number of freeze-thaw cycles of the accelerated weathering test and the number of years of natural weathering. Based on the studies performed so far, it can be assumed that between 12 to 16 cycles of the accelerated weathering test procedure is equivalent to one year of natural weathering of a stone facade in a northern temperate environment.

This information can be further used in determining more accurate safety factors for a stone facade rather than the empirical safety factors that are recommended by the stone industry. Our experience has shown that in certain

situations a lower safety factor may be acceptable that can provide a more economical stone thickness or anchor spacing. This can reduce the overall cost of a stone facade.

A designer must consider a particular stone can vary from quarry to quarry, from different locations within a single quarry, and possibly within a single block of the quarry. Therefore, it is important to test the specific supply of stone for a large building project. Stone used successfully on a similar building project in the past may not have the same physical or mechanical properties for the current building project.

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60 SURVEY RESULTS REGARDING ATTITUDE TOWARDS STONECLEANING OF BUILDING SANDSTONES

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Abstract

Over the past 20-35 years, cleaning of stone facades of buildings has been a significant activity, both in terms of financial outlay and the effect on the built heritage of Scotland's cities. The removal of soiling has been perceived by the general public and building owners as a "good thing" because of the simplistic notion that a clean, bright facade reflects well on the urban environment in general and on the image of the building occupier in particular. However, the potential for immediate and longer term damage to stone is seldom fully appreciated. Both abrasive and chemical cleaning methods can cause changes to the physical and chemical characteristics of sandstones which may increase the rate of decay of a previously cleaned stone. These effects may take a number of years to manifest themselves and it has recently become evident that stone replacement is being carried out on facades that have been cleaned in the past. This research programme was initiated by Historic Scotland to investigate the scale of any problem with respect to the decay of building sandstones. A database of information is being collected to determine the relative rates of decay of cleaned and un-cleaned sandstone facades. This will allow estimates to be made of the amount of stone required for repair purposes and will provide projections with regard to future repair costs. Objective and quantifiable evidence on the impact of stonecleaning with respect to stone decay and stone replacement is now urgently required if proper guidance is to be provided, to both practitioners and the stone industry.

Keywords: database, sandstone, stonecleaning, stone decay, stone repair.

1 Introduction

In the previous two decades, many of the mainly sandstone buildings in Scotland's cities have undergone stonecleaning. Where figures for grant aid have been collected, the amount spent on stonecleaning has been estimated at about £9M per year (by Scottish Development Agency for the three years to 1992). The large amount of cleaning which has taken place has had a significant effect on the appearance of buildings and on their environmental context. Stonecleaning is generally perceived by the public and building owners as a "good thing" because of the simplistic notion that a clean, bright facade reflects well on the urban environment and on the image of the building occupier. However, no stonecleaning method has yet been devised which can remove soiling without affecting the underlying stone. Physical cleaning methods such as grit blasting inevitably result in some abrasive damage to the stone surface. Chemical cleaning may dissolve some stone components along with the soiling and can leave substantial amounts of chemical residues in porous stone. The soiling layer is often hydrophobic in character and its removal is likely to change the behaviour of the stone (and joints) with respect to movement of water and water vapour.

Where the soiling layer is responsible for accelerated decay of the underlying stone then cleaning may be advantageous. However, stonecleaning is often carried out for purely aesthetic reasons with little understanding of the potential damage which may be done by improper or uncontrolled cleaning. The amount of damage caused at the time of cleaning varies depending on the technique and the stone characteristics, but what is seldom understood is that stonecleaning has consequences for the longer term behaviour of the stone. Previous studies (Andrew et al. 1994; Urquhart et al. 1993, 1994; Webster et al. 1992; Young & Urquhart 1998; Young 1998) have established that stonecleaning can cause immediate damage to sandstone buildings and that it has the potential to accelerate decay in the longer term. However, no previous studies have attempted to quantify the scale of this damage. This research project aims to determine the extent and rate of stone decay on stonecleaned sandstone facades relative to un-cleaned facades and to predict the effects of previous and on-going stonecleaning on the future costs of facade maintenance. Objective and quantifiable evidence on the impact of stonecleaning with respect to stone decay and stone replacement is now urgently required if proper guidance is to be provided, to both practitioners and the stone industry.

2 Methodology

One of the objectives of this research programme is to predict the effects of stonecleaning on the future costs of maintenance. As a first step, it was therefore necessary to establish the motivations behind current stone repair and replacement. A questionnaire survey was used to determine the extent of awareness among practitioners of the issues and problems associated with stonecleaning and the replacement and repair of damaged or decayed stone. A total of 100 questionnaires were sent out to architects, developers and planning departments of City Councils. Answers were received from 42 respondents. In this way, preliminary information was gathered on the frequency of maintenance interventions, attitudes regarding

maintenance methods, the choice of material, costing methods in use and any prior knowledge of respondents regarding possible negative effects of stonecleaning.

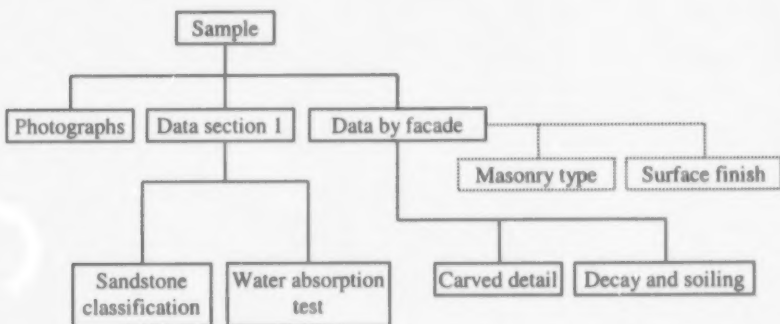
This initial survey has been followed by a series of more closely focused investigations of cost, specification and materials sourcing. The life cycle cost model which is currently being developed (Laing et al. this volume) reflects the link between stone decay (resulting from stonecleaning) and remedial work. The point at which such work would be completed in reality is vital to both the cost and resources required. Information was gathered in the following stages:

1. Initial costs of cleaning (including temporary works and testing) represent a major financial cost. Previous work has shown that aesthetic (Webster et al. 1992) or financial (Laing and Urquhart 1997) benefits may result, at least in the short term.
2. Where accelerated or additional stone decay results from stonecleaning, this might require maintenance work. The extent and rate of decay are to be established through the compilation and analysis of a buildings database.
3. Parties responsible for the specification of stone maintenance were questioned regarding both stonecleaning and stone repair with respect to cleaning methods, reasons for cleaning, reasons (quantified) for stone repair or replacement and sourcing of materials. Linked with information from item 2. above, the timing and extent of maintenance work during the life cycle can be established.
4. Stone masons were questioned regarding the techniques and costs of stone repair and replacement work. In addition to financial information (e.g. unit rates), data was also gathered regarding labour and materials costs. It was hypothesised that the often long timescales involved could reduce the apparent scale of work after discounting had taken place. By reporting the resource implications there was less danger of this occurring.
5. Work has begun on the investigation of the quantity and quality of stone available for repair and replacement. Issues concerning skills required, variables used in matching of stone and the practicalities of sourcing stone from closed or mothballed quarries will be covered. This section of the work, while clearly following the other areas, will have important implications for the study as a whole.

Over 200 buildings located in various Scottish cities have been identified for inclusion in a survey to determine the relative rates of decay on cleaned and un-cleaned sandstone facades. Most buildings have been chosen as pairs of essentially identical cleaned and un-cleaned facades to allow realistic comparisons to be made of their condition. Wherever possible the pairs of buildings are in the same terrace (often side-by-side) to eliminate the effects of local variations in environment. The facades chosen cover a wide range of stone types and cleaning methods. A database is being constructed to hold and analyse the information collected from field visits to survey the condition of these buildings. Analysis of information regarding the mapping of decay, soiling and interventions (e.g. repair) on these facades will allow estimates to be made of the amount of stone required for repair purposes and will provide projections with regard to future repair costs.

Table 1: Database structure

Table name	Parent table	Position in hierarchy	Brief description of contents
Sample (S)	-	Primary parent	Address, reference number and all details known about the building prior to survey, including architectural description, type of sandstone and cleaning method.
Photographs	S	2nd level	Facade photographs with the facility to print survey forms.
Data section 1 (DS1)	S	2nd level	All data pertinent to the building as a whole including its relationship to its environs and neighbours.
Sandstone classification	DS1	3rd level	Description of sandstones.
Data by facade (DbF)	S	2nd level	Facade specific data including assessment of architectural and ornamental detail and general facade condition.
Masonry type	DbF	sub-table	Masonry type used on the facade.
Surface finish	DbF	sub-table	Type of surface finish used on the facade.
Carved detail	DbF	3rd level	Photograph and assessment of complexity of carved detail.
Decay and soiling	DbF	3rd level	Percentage coverage of the facade by different degrees of soiling, soiling types, biological growths, colour changes, decay types and repairs.
Water absorption test	DS1	3rd level	Results from field tests.



The database is comprised of a number of related data tables whose contents and relationships are outlined in Table 1. More detail about the nature of the essential core data is contained, where relevant, in Table 2. As can be seen from the contents of these tables it is possible to account for a considerable number of variables. The essential comparison will be of the differences in rates of decay between cleaned and un-cleaned buildings in similar locations of similar architectural complexity. Other important comparisons include the amount of stone replacement and repair, soiling rates and amount of biological growth.

Table 2: Nature of essential data collected

Type of data	Description	Data table
Sandstone description	Cement type(s), colour, grain size, sorting, grain shape, constituents, fabric, bedding depth and source.	Sandstone classification,
Environmental details	Orientation and alignment of terrace, road category, proximity to road, traffic density, relationship to neighbouring properties, layout density, terraced or not, shape of terrace, position in terrace.	Data section 1
Treatment of facade	Cleaned or un-cleaned, date and method of cleaning.	Data section 1
Physical appearance	General appearance, condition of visible rainwater goods, staining from hidden rainwater goods, condition of joints and beds, crispness of detail and arises.	Data by facade
Soiling characteristics	Density and pattern of particulate soiling.	Decay & soiling
Crusts and deposits	Crusts (e.g. gypsum), salt efflorescences and bird droppings.	Decay & soiling
Biological colonisation	Extent & location of colonisation by algae, lichens & higher plants.	Decay & soiling
Colour change	Natural mineral movement, chemical bleaching or staining, run-off from metal fixings, splash-back and water run-off patterns.	Decay & soiling
Decay and repair	Granular disintegration, blistering, flaking, contour scaling, solution decay, abrasion, decay of components, delamination, severe pitting, irregular pitting, differential decay, mechanical damage, fissures, debasement of detail sharpness, redressing, plastic repair, lack of mortar, inappropriate bedding, indenting, honeycombing.	Decay & soiling
Survey conditions	The weather conditions are noted at the time of survey as this can have an effect on what the surveyor is able to detect (particularly in the case of salt efflorescences).	Data section 1

3 Results

3.1 Questionnaire survey

Using a series of photographs showing different types and degrees of decay, respondents were asked to indicate whether or not they would replace or repair the stone and to indicate their reasons: aesthetic, functional or structural. The answers varied markedly for the different examples provided which covered a range of stone conditions from slight damage to severe decay. In many cases, the reasons cited for stones requiring replacement or repair were primarily aesthetic (Fig. 1). Problems with the stone's ability to function (e.g. in water shedding) were cited rather less often and relatively few respondents thought the examples of decayed stones represented a structural threat, although this depended on the state of the stone and its perceived role in the facade. Where stone decay was perceived as a structural threat, replacement was preferred over repair.

Respondents were asked what time scale they considered reasonable for carrying out particular forms of maintenance on a natural stone building facade (Fig. 2). The only maintenance interventions that all respondents agreed should be carried out at some stage were "re-pointing" and "replacing decayed stone with natural stone". For all the other interventions suggested, at least some respondents were of the opinion that they should "never" be carried out. The most rejected intervention was "application of

water repellent" which 69% of respondents felt should never be done. As many as 50% of respondents replied that "stonecleaning" should never be carried out. Biocide application and plastic repair were also rejected by a large number of respondents (43% and 33% respectively).

The logistics and cost of stone sourcing will be investigated as they relate to the immediate and future needs for matching stone for repair purposes. With regard to gathering information which can be used to inform further studies, respondents were asked to what extent they regarded particular items (Fig. 3) to be of importance when preparing a specification for matching replacement stone to original stone. Responses were all greatly skewed towards the higher end of the scale (more importance), indicating that respondents perceive the correct selection of material to be of great importance.

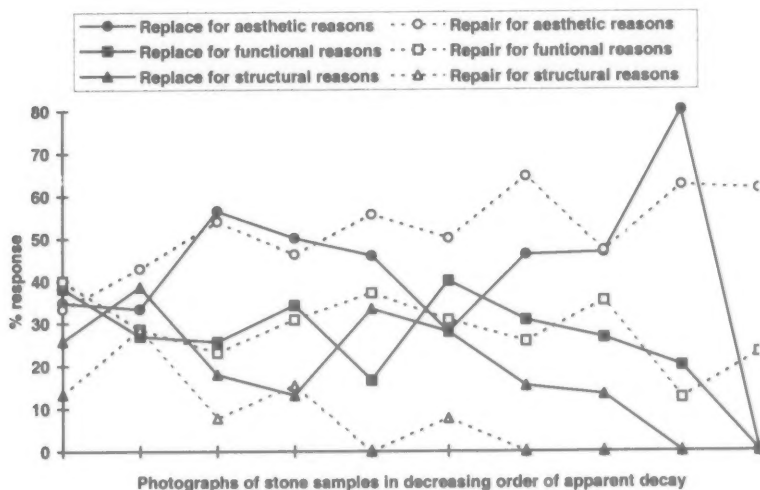


Fig. 1: Percentage distribution of reasons given why a stone requires replacement or repair, from those respondents who thought each given stone did require some intervention

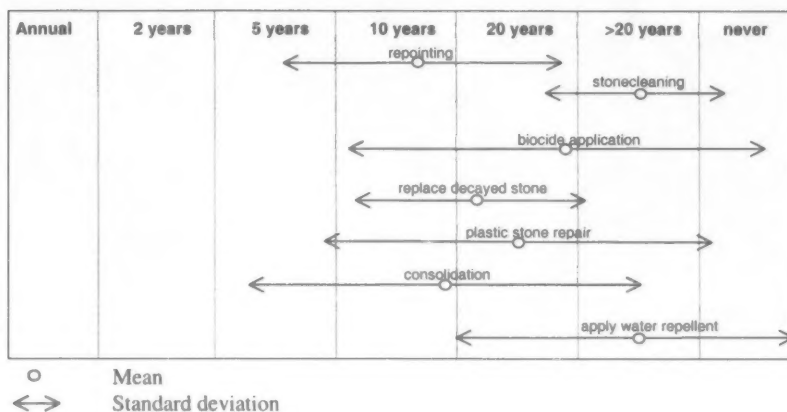


Fig. 2: Respondents' answers to the question "What time scale do you consider reasonable for carrying out the following maintenance on a natural stone building facade? "

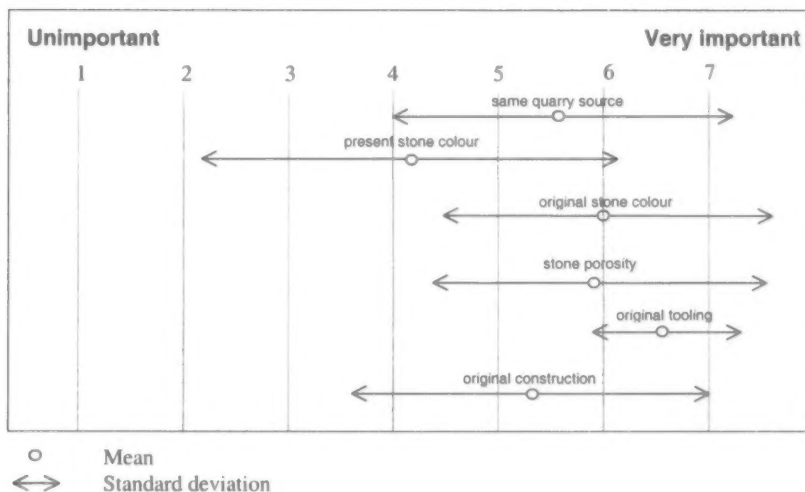


Fig. 3: Respondents' answers to the question "When preparing a specification for matching replacement stone to original stone, to what extent do you regard the following to be of importance? "

Perhaps surprisingly, the item respondents thought to be of most importance in matching replacement stone to original stone was "matching the original tooling", 67% of respondents rating this as very important. While only 12% of respondents rated "matching present stone colour" as very important, 45% rated "matching original stone colour" as very important. This may be an indication of the general appreciation of respondents that stone colour changes throughout the life span of a building and that initial colour differences of replacement stone need not be seen as a problem. In general, with the exception of present stone colour, all the items specified were seen as moderately to highly important by respondents.

Respondents were asked whether they had noticed any detrimental effects to natural stone building facades caused by chemical or abrasive stonecleaning. The prior knowledge of respondents is interesting in two main respects. Firstly, the responses to other parts of the questionnaire might be influenced by such knowledge and, secondly, the depth of knowledge informs as to the extent to which previous research findings are currently recognised at a practical level. The overwhelming majority of respondents (86%) had observed a variety of detrimental effects of stonecleaning (Table 3). Erosion was overwhelmingly the most commonly noticed detrimental effect, being cited by 44% of respondents. 10-20% of respondents had also noticed increased biological growth following cleaning, discoloration, surface spalling and salt efflorescences. Accelerated weathering was only remarked upon by one respondent.

Table 3: Number and percentage of respondents citing particular detrimental effects of stonecleaning that they have noticed on building facades

Observed effect	Number of responses	% response
Erosion	18	44
Increased biological growth	7	17
Discoloration	7	17
Spalling/scaling	5	12
Efflorescence	5	12
Pitting	3	7
Bleaching	2	5
Accelerated weathering	1	2
Damage to adjacent buildings	1	2
Delamination	1	2
Increased porosity	1	2
Joints opened up	1	2
Loss of cement	1	2

3.2 Database development

At the time of writing, survey information is being entered into the database but no formal statistical analysis has as yet been carried out. However, early indications have suggested that, in some circumstances, a significant difference is likely to be found in the rates of decay between buildings which have undergone stonecleaning and those which have not. A fuller understanding of any link between stonecleaning and rates of stone decay will only become possible following a full analysis of the information once the data collection phase of the project is complete.

4 Discussion

Continuing work on this research programme involves the completion and analysis of the database and follow-up interviews to the questionnaire survey.

If it can be shown that stonecleaning causes an acceleration of decay then there are cost implications for the maintenance cycle of the fabric of the building. Results from the questionnaire survey indicated that the primary reasons for most stone replacement and repair were aesthetic. This implies that stones on which decay (including that caused by stonecleaning) is relatively superficial and poses no immediate structural or functional threat to a building's performance may be frequently considered suitable for repair or replacement during facade maintenance. Even a relatively small degree of accelerated stone decay as a result of stonecleaning may therefore greatly increase that amount of repair and replacement over the following years. The implication here is that accelerated rates of even superficial decay will lead to higher costs. It is clearly essential that the long term effects of stonecleaning methods are fully appreciated by building owners and professionals involved in building maintenance before any decision to clean a facade is taken. Respondents answers to the questionnaire indicated that many were wary of the long term implications of irreversible treatments to stone (including stonecleaning, biocide application and use of water repellents). This response is clearly justified where there is a lack of useful practical guidance regarding the long term effects of treatments. Industry needs to be informed by "best practice" and best practice needs to be supported by research.

The existence of any link between accelerated stone decay and particular stonecleaning methods carries major implications for:

1. an appraisal the desirability in stonecleaning *per se*,
2. a re-assessment of current cleaning methods and their suitability for certain types of stone,
3. interpretation of how the consequences of stonecleaning may affect the maintenance cycle of buildings and the production of cost models.

5 Acknowledgements

This research was commissioned by Historic Scotland.

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PART THREE

**DURABILITY OF WOOD AND
WOOD COMPONENTS**



61 THE PROBLEM OF THE DURABILITY OF MATERIALS USED IN THE REHABILITATION OF HISTORIC TIMBER ROOFS

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Abstract

This paper focuses on the specific problems connected with the repair and refurbishment of historic timber roof structures. Modern materials used to improve the overall performance of the roof and to guarantee the durability of the work are analysed, focusing in particular on waterproof membranes. Among these materials one is examined in detail: the open roof lining Tyvek®¹ vapour barrier, the properties and the behaviour of which are examined and compared with other coatings more commonly used for this purpose. The paper concludes with the description of a case study of application, that is still in progress: the refurbishment of the roof of the 19th Century extension of the Valentino's Castle in Turin (Italy).

Keywords: Timber, Structures, Refurbishment, Waterproof materials, Durability

1 Introduction

This paper discusses the work of repair and refurbishment on historic roof structures and analyses the various approaches and methods for maintenance, repair and, conservation and restoration, focusing mainly on timber structures. The issue of durability of structures and their repair calls for a discussion on the thorny matter of maintenance and the meaning of that which lies at the borderline between either activity. Which materials, techniques and systems can guarantee the required reliability and other criteria, such as reversibility and compatibility?

Field specialists (i.e. conservation architects, engineers, building heritage

¹ Tyvek is DuPont's registered trademark.

conservation officers, etc.) are fully aware that any conservation work on a historic building starts from the roof system, as the performance level of this element is essential to ensure the conservation of the entire building. Very often however, the roof is considered an element of the building that can be "sacrificed", regardless of its actual condition in regards to conservation and its capability to perform its role effectively. Thus, very often work on roof structures is entrusted directly to contractors that are not under the direct control of the designer in charge of the building conservation and restoration works. In this situation, many roof structures and protective parts (e.g. roof boarding, roofing) are not subjected to inspections or diagnosis and are replaced with new roofs. This in turn potentially brings about the danger of losing important parts of the Italian building heritage.

2 The central role of the diagnosis: inspection and assessment of the state of conservation.

In the past few years there has been a renewed interest in protecting the culturally rich - though little known - Italian historic and heritage timber structures. Special consideration has been given to timber structures in roofing systems found in monumental and traditional architectural complexes, that have only recently been acknowledged as a part of the cultural heritage.

The problem of conserving these items calls for a correct *in situ* diagnosis that includes historical and architectural analysis and, with regards to material and structures, evaluating the state of preservation and structural efficacy. One of the objective difficulties when restoring these structures is to evaluate the state of preservation of the material and the acceptable performance of structural timber elements *in situ*. This difficulty is all the greater in the case of roofing elements, owing to problems of restricted access for carrying out inspections and tests (Bertolini Cestari 1998; Bertolini Cestari and Curtetti 1998; Bertolini Cestari et al. 1998; Bertolini Cestari et al. 1999).

The diagnosis of timber structures must always be based on a project divided into two distinct analytical levels: the first level consists of a visual inspection performed by technicians using simple instruments to identify the wood species, the thermohygrometric conditions and the natural or artificial durability; the second level of diagnosis employs appropriate instruments to obtain information that cannot be deduced from the first inspection level.

Instrument-aided diagnosis for full-size timber structures has not yet reached a correct level of procedural organisation and nor have reliable techniques evolved to assess either the state of conservation or the load-bearing capacity. Consequently, only when these techniques are adequately widespread among operators will they contribute to the conservation of many timber structures, rehabilitated and conserved in static terms, by works that fully respect both the original conception and the historical-technological role of the structures, as well as the material with which they are constructed (Bertolini Cestari 1994; Bertolini Cestari and Cravero 1998; Bertolini Cestari et al. 1995; Bertolini Cestari 1997; Bertolini Cestari et al. 1997).

The two levels of diagnosis identified by Italian experts (Bertolini Cestari et al. 1998) are outlined in the following paragraphs.

2.1 Visual inspection (*first level diagnosis*)

The *first level* is aimed at a general evaluation of the entire structure without an accurate analysis of the individual timbers. This level helps orient the second, by identifying the specific needs and priorities for further inspections. The general evaluation requires simple tools for investigating the wood and appropriate conditions for the inspection, i.e., accessibility to the structures, sufficient light and clean surfaces. Furthermore, the availability of a plan of the structure is essential for mapping the information and for identifying classes of priorities for the next step.

2.2 Instrument-aided inspection (*second level diagnosis*)

The *second level* is based on the information provided in the first level. It entails a detailed evaluation of the condition of each timber and the joints within the structure. It requires the same general conditions and involves a thorough visual inspection of the timber characteristics and a more sophisticated instrument-aided analysis: e.g., moisture measurements of the wood, microscopic analysis of fragments for the identification of the species, micro-drilling analysis, etc. Among these, the use of a portable micro-drilling instrument (e.g. Resistograph® or DDD200®) is particularly interesting for assessing the state of conservation of the material. The joints are subject to a detailed assessment at this level of analysis, especially where previous restorations introduced other materials, so as to evaluate and monitor the compatibility with the original wood. The second level of analysis is translated into a comprehensive and - where possible - quantitative mapping of the conditions (in terms of resistant cross-sections of the timbers) as a basis for the structural calculation (Bertolini Cestari 1998).

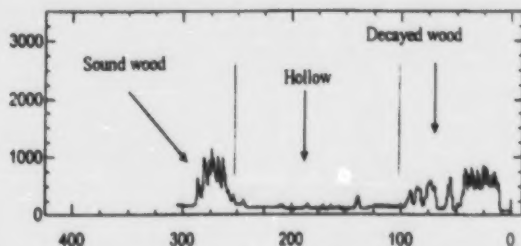


Fig. 1: Resistographic plot in relation to depth of penetration in cm (starting on right) on an apparently sound piece of timber. The plot shows, after 4 cm of sound wood, a hollow portion taking up almost all the section, excluding the last 3 cm. It is also possible to evaluate that the original section was about 26 cm.

3 Consolidation techniques of roof structures related to structural damage

3.1 General criteria

In current technical literature, certain works - included those with innovative features - appear strongly delayed with respect to the conservative criteria which are being imposed for monumental and historical building restoration or more simply, by tradition. Too often we witness operative practices that prevalently address radical works: timber ceilings consolidated with reinforced concrete mantles replacing all or part of the structure; total or partial reconstruction of roofs with steel or glued wood laminates. The introduction of materials extraneous to the timber restoration tradition indicates a wide-spread loss of building know-how on behalf of designers and sector workmen. For example, Sironi (1994) suggests a work on a timber beam ceiling with incredible proportions for an ancient structure. The base-to-height ratio (8 x 20 cm) is closer to that of a laminar beam structures than that of solid wood and the cut resistance bolts lead to perplexity on the possibility of insertion without breaking the timber beam extrados. Moreover, the work prescribes L-brackets, stands and suspending brackets. Is all this imagination actually necessary?

With these premises, which techniques and technologies and which materials can be usefully recommended today for consolidation works?

Currently, a designer involved in a structural work on ancient manufactures cannot avoid being stricken - and even intimidated or bewildered - by the great variety of recently performed structural works on a wide range of structures that include ceilings, frames, pillars, and trusses, quoting the most common structures. Moreover, the design suggestions emerging from the flourishing technical literature in this sector too often represent very particular cases and are treated as unique. The proliferation of various consolidation technique "patents" - consisting of steel sheets and wires placed in the core of the beam, multiple layer techniques, epoxy resin applications, etc. - appear to provide the catalogue of solutions and techniques to be adopted to solve the problem (Tampone 1994; Asti 1994).

In this scenario, is providing indications to address the choices on methodological and practical level still meaningful? On the other hand, the conviction that no suggestions in principle and in practice can be offered would fatally lead to restoration works being conceived one at a time or even - for the most complex cases - step-by-step, with unplanned consequences in execution and decisional adaptation.

This paper certainly does not intend to treat the complete, complex and delicate issue of structural consolidation of timber manufactures neither does it intend to offer final solutions to problems which underwent complex and sometimes contradictory evolutions. The solutions appear to be tending to more mature and balanced insertions in the picture of wider general structural restoration¹. The starting point,

¹ UNI committee on "Recommendations for the Structural Restoration of Civil Buildings and Architectural Heritage" is preparing a paper on the technical analysis, work technologies and materials to be used to restore ancient buildings.

however, is understanding the existence of many shortcomings and the conviction that much can still be done to eliminate the problems on methodological and practical level. With this reference, this paper aims at offering a contribution regarding the main parameters which take place in choosing from the most common work techniques.

3.2 Restoring reinforcement and consolidation techniques

In the wide and articulated case study of current carpentry restoration-consolidation, a schematic subdivision can provide an initial orientation. Timber structure consolidation can be classified by type according to the works required and to the instability found. Schematically, two major types can be identified: localised works and works on the global structure.

The first is the consequence of existing deterioration on single mouldings or more commonly near panel points with other mouldings (internal connections) or at coupling points with surrounding walls (external connections) where conditions of moisture and poor ventilation at beam headers causes biological attacks on materials.

The latter type of work prevalently involves the need to adapt to changed conditions of service and, consequently, addresses structural safety issues.

This schematic organisation highlights the different purposes of the two types of work and specifically:

- Restoring or rebuilding works on material considerably altered or damaged by external factors (biological attacks, fire, demolition, etc.). Localised reconstructive techniques include *in situ* fitted wood prostheses, glued sheets, epoxy cement.
- Reinforcement works with substantial changes to the existing system tending to increase the resistant capacity of the original structure. This category of works on trusses includes positioning metal tie-rods, adding steel ribbing and inserting fibreglass bars to consolidate connections.

It is important to note how these two types of works are referred to panel points and trusses stiffening couplings. Essentially, the works change the structural arrangement of the truss with reference to stiffness ratio at local and complex level. From a static point of view, these alterations can produce qualitative and quantitative tensional alterations which cannot be neglected.

The following paragraph illustrates and discusses a work technique that causes alterations and changes to the structural arrangement.

4 Principle of work on historic timber structures

Interest in Italy in the conservation and restoration of ancient timber structures, important aspects of our architectural heritage, represents a recently introduced sector of the Cultural Heritage Department, raising problems regarding diagnostic surveys and work techniques focused on conservation. Over the past few years, there has

developed a firm conviction that these structures must - wherever possible - be conserved and restored in accordance with their static role, using works that are consistent not only with the original design but also with the material: timber.

However, in practice, many recent works involving structural restoration have betrayed the idea of conservation: in extreme instances we have witnessed the unjustified demolition of century-old floors, coffer ceilings and roofs; more frequently, ancient buildings have been altered by the widespread use of new structures and new materials in replacement of the original ones.

These arbitrary, and to say the least "heavy-handed", works are in the majority of cases induced by the difficulty of evaluating the state of the conservation of materials and the real load-bearing capacity of the structural elements, or by an incorrect evaluation of the overall structural function, or the choice of summary and profitable procedures rather than procedures guided by the real needs of the timber components.

It is precisely their load-bearing capacity that is constantly debated today, in spite of the renewed interest in studies on wood, by operators who, clearly lacking the necessary know-how, demonstrate their complete distrust towards existing timber structures with their works.

This distrust has been expressed and given tangible form through the development of consolidation techniques that foresee the extensive inclusion of various types of implant, made from "new" materials that, precisely because they are new, are regarded as superior from a short-sighted point of view that does not take account of medium and long-term behaviour: this means *problem of durability*.

5 The duration of refurbishment: the roofing system

We will specifically analyse the example of discontinuous roofing. Discontinuous roofing is a type of roofing where the sealing elements ensure the required waterproofing features only in the presence of roofing surface slope values exceeding a minimum fixed value, according to the product used. Such roofing presents a variety of heterogeneous functional models, products, materials and technologies.

The sealing element - one of the elements that most characterise the roofing - can present a range of different shapes, dimensions and materials, as well as various versions from a functional point of view. Some products ensure waterproofing with appropriate longitudinal and transversal superposition (e.g. tiles), for which the slope is essential, while other products are characterised by presenting the same length as the roof pitch. In this case, the sealing along the lateral superposition must be guaranteed and the slope is not as essential, as its main purpose is to ensure a regular flow of water.

The refurbishment work is often connected with re-use of the building for current human activities and for modern comfort requirements. Thus, heating and air conditioning plants are generally installed and higher temperatures are required. Attics which, in the past, were in most of cases used for storage are nowadays

enhanced and converted into liveable spaces. Consequently, better thermal insulation must be provided and new problems of moisture distribution arise inside the roofing system, as higher quantities of water vapour are present in the inside air. Commonly, a vapour barrier is introduced at the bottom of the insulation layer to prevent water condensation in the structure and a membrane is placed at the top of insulation as an additional protection from water leaking under the tiles due to poor roofing maintenance.

Unfortunately, in many cases, this solution - which is theoretically sound - creates a range of problems:

- the vapour barrier allows moisture through if it is not correctly laid;
- the ageing of the barrier, or tampering in time, creates gaps allowing the access of moisture.

In both cases, the vapour is trapped under the insulating material and can lead to:

- increased moisture in the underlying timber structures with possible rotting;
- decreased overall thermal resistance of the roofing;
- condensation, with the formation of stains and surfacing on the internal side of the roofing.

The most up-to-date suggestions to prevent these problems address the use of membranes, that are waterproof yet permeable to vapour, placed on top of the insulating material. Furthermore, an under roofing ventilation may be considered, even if the suitability and the modality of this sort of intervention is currently debated (Künzel and Grobkinsky 1989).

The industry has developed several types of permeable membranes offering a wide range of mechanical and permeability features. These materials include Tyvek®, which provides good tear resistance and vapour permeability, with permeability considerably higher (~ 400X greater) than those offered by the more common micro-pore polyethylene film membranes.¹ These features appear unaltered over the medium to long term. This membrane was chosen for the present intervention (see Fig. 2).

¹ BBA (British Board of Agreement) Agrément Certificate No 90/2548, No 94/3054.

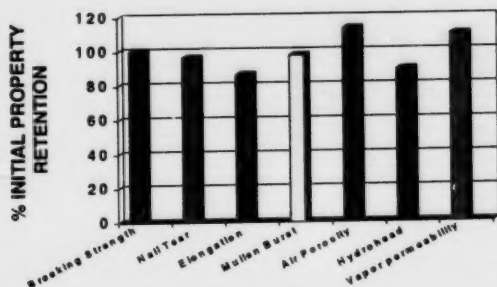


Fig. 2: Tyvek behaviour after 16 years: % initial properties¹

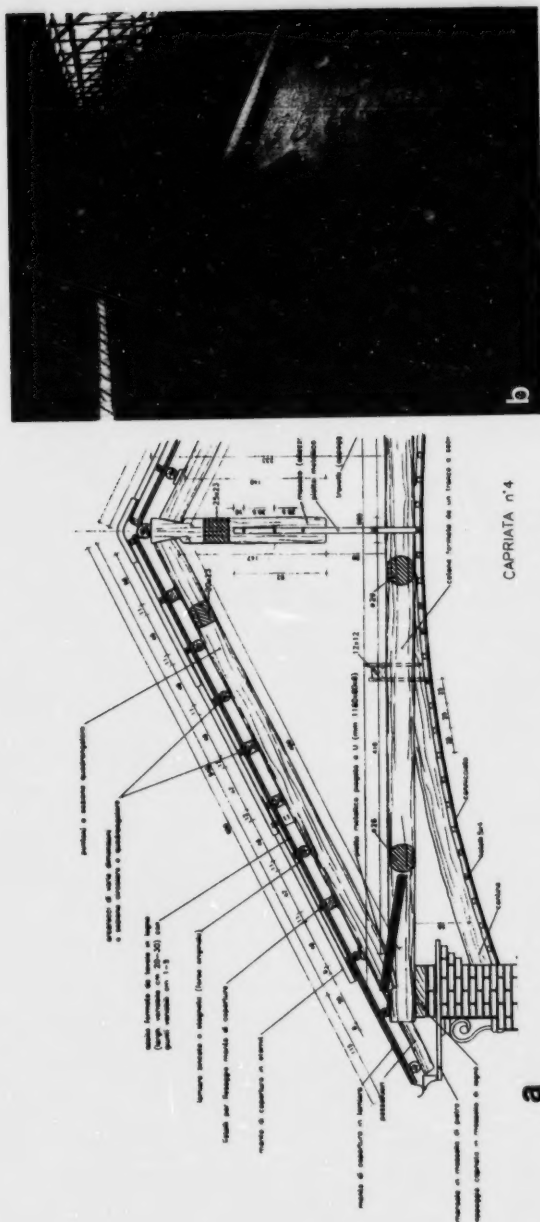
5.1 A case study: Valentino's Castle

The following case study analyses the reconstruction work on the roofing of the Valentino Castle in Turin. The roof covering the side extensions facing the central courtyard of the Castle was built during the second half of the last century. According to historic sources, the original roofing was made of zinc or tin plated metal sheets. There is evidence to confirmed this: fragments of this material can be found between the roof-boarding and the asbestos cement sheets put in place during this century.

The side extension presents a double-pitch "Eternit" asbestos cement roofing. The plan dimensions are approximately 44.60 x 11.60 m, the eaves height is 13.00 and the roof ridge height is 16.00 m. The load-bearing structure of the roof consists of 16 timber trusses arranged with a variable middle line distance from 2.50 to 3.00 m. Longitudinal spines support a continuous boarding which is nailed to the spines (Fig. 3). The pitch boarding presents a rather irregular shape and projects from under the layer of "Eternit" - forming the eaves weathering and reaching the gutters.

The metal sheet is probably the most ancient pitch covering. It was not removed during the work carried out in the '60's and the Eternit panels were directly placed over it. The administration of the Polytechnic of Torino decided to replace the roofing with a new material. A specialised investigation was carried out to evaluate the state of conservation of the material, the species of the wood, the physical-mechanical features and the attacks of bacteria and micro-organisms. The results of this diagnosis confirmed the good state of conservation of the timber carpentry and the suitability of the wooden boarding to support the new panelling.

¹ This diagram is kindly given by Du Pont Engineering Products SA



Consequently, as agreed with the Cultural Heritage Department, the choice was directed to zinc metal sheets offering special aesthetic features (being the product used for monumental buildings) and, above all, the required resistance and durability. More in detail, the material consists of 8 mm thick, pure, 99.995% copper and titanium alloy, electrolytic zinc laminate (e.g. Rheinzink®) presenting good features in terms of machining and resistance to atmospheric agents. The covering is fastened by means of a continuous boarding and the jointing consists of a double vertical seaming, i.e. longitudinal seaming of bands arranged off the water flow surface.

The proposed system devotes special care to ventilation under the zinc covering. Ventilation is ensured in the volume of area under the roof without the need to create additional ventilated layers and vapour barriers. The issues related to complete waterproofing, on one hand, and to vapour transpiration features, on the other, for this roofing system led to employing a under tile material which could join the two features (i.e. impermeable and transpiring).

6 Conclusions

The paper offers an overview of the problems connected with the durability of the rehabilitation work specifically on historic timber roofs, and describes a work in progress for which the durability request is intended obtained with the use of a under roofing vapor open membrane.

7 Acknowledgements

This study refers to the results of a research "Diagnosis and Evaluation of the Loadbearing Capacity of Timber Structures: Optimising Structural Checks on Products Once Part of Historical Buildings" (U. O. Co-ordinator: C. Bertolini, in a strategic CNR-Cultural Wealth Supervising Agency, 1996-2000).

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62 DURABILITY OF RENOVATED AND PAINTED WOODEN WINDOWS

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Abstract

For reasons of sustainability great efforts have been made to re-use old wooden windows, making windows from old wood and treating wooden windows with environmentally friendly paint without the use of wood preservatives. The paper presents results from projects of the, above, mentioned type in which the Danish Building Research Institute (SBI) has been involved.

The windows have been assessed for performance properties like tightness to wind and water, for old windows after renovation. Further, the long-term properties have been assessed by investigation of the influence from artificial or natural ageing. The assessment was based on sign of deterioration indicated by spalling, cracking, blistering etc. in the surface. Besides, in some cases the moisture uptake during artificial or natural ageing has been used as an indicator of the potential risk of fungus and dry rot.

As expected quite considerable changes in the durability were recorded. Some of the findings, were however, quite surprising. Normally in Denmark it is expected that old wooden windows are made from much better wood than modern ones and that they, for this reason, last much longer. This was found not to be a general valid statement. Also, it was quite remarkable that only minor differences in the deterioration of painted and unpainted windows were noted.

As regards painting, it has been attempted to find alternatives to the use of wood preservatives. Efforts have focused on the use of linseed oil, which is well known from old buildings. However, the painting process using of linseed oil is quite complicated, and besides several dogmas as to the use of linseed oil exist whereas scientific evidence is lacking. Different paints - both traditional and based on linseed oil - and substrates have been tested and it appears that the quality and pre-treatment of the substrate and the workmanship are crucial to achieve a good result.

Keywords: Windows, durability, wood, paint, renovation.

1 Introduction

In Denmark windows have always been and still are most commonly manufactured from wood. Old wooden windows are for this reason quite normal and consequently there is a big interest in methods for maintaining the windows. Due to the interest in sustainability much more interest has for the last decade been paid to the possibility of re-using wooden windows and using paints or wood preservatives that do not harm the environment.

Paints, wood preservatives and other surface coatings constitute a serious problem to the built environment. Earlier paints contained fairly big amounts of VOC's which caused big occupational health problems to the painters. Further, corrosion protections and wood preservatives contained lead and other heavy-metals which are harmful to the environment.

When the water based paints entered the market the occupational health problems were considerably reduced. On the other hand the quality of the paint, especially as regards outdoor use has been a problem. The water based paints are softer in the surface and there have been reported quite a few examples of fungus and dry rot in windows painted with water based paint where in contrast not-maintained windows with old peeling paint did well. This is considered to be due to the fact that the paint constitutes a skin on the surface and once water has penetrated behind the skin it can only escape by diffusion which is a slow process.

In contrast to the wish of using "green" products stands the need for more efficient wood preservatives as it is the impression that the wood currently used for windows does not match the quality of earlier times. The wood is from more rapid growing trees giving softer and more open wood and, besides, it is cut and treated in another way than before, which also affects the quality negatively.

2 Testing of windows for "the recycled house"

In a Danish project entitled "the recycled house" it was the intention to construct a complete house from re-used materials. As part of this project an investigation of wooden windows was performed. The objective was to assess the possibilities of re-using windows. These could either be old windows from demolitions, which were renovated and re-used, or new windows made from old wood, e.g. timber members from demolished houses.

The purpose of the investigation was to test the performance, i.e. the airtightness and the watertightness of the windows, and the deterioration when exposed to artificial ageing.

In the following observations from the test are described to illustrate problems that may arise using recycled windows or windows made from recycled wood.

2.1 Re-used window

For this investigation a traditional Danish wooden window with four outward opening lights was tested in three different versions.

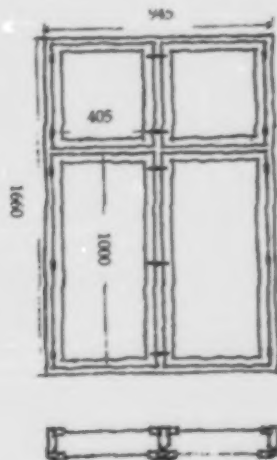


Fig. 1: Front view and cross section of wooden window for the "the recycled house"

2.1.1 Window with additional frames attached to the inside of the frame

In this version the window was tested with additional opening sashes mounted on the inside of main frame. A sketch of the window is shown on Fig. 1. At the receipt of the window it was noted that the sashes did not fit the main frame properly so that the overlap in the rabbet, at worst, was diminished to one half of what it ought to be. Hinges and screws were partly corroded especially in places where new screws had been applied and finally the upper sashes sloped the wrong way, i.e. inwards, presumably because the sashes had been turned upside down at renovation.

At testing the window showed an unacceptable airtightness and watertightness. To improve the tightness wooden sealing lists were mounted on the main frame and the vertical mullion. This resulted in a satisfactory airtightness whereas the watertightness still was unacceptable. Water penetrated in considerable amounts at the top sashes, because they sloped inwards. A drip cap, which was subsequently attached to the top of the main frame, could only to some degree reduce the amount of water entering between the outer frame and the sash.

2.1.2 Re-used window with one layer of glass

The window for this test was the same as the first one only was the additional sashes removed and, subsequently, additional sealing lists were mounted on the horizontal mullion as well, i.e. there was sealing all way round the sashes on the inside. Further, the old hasps were replaced by a newer model, which resulted in the window being tightly closed.

The test showed that even if the airtightness was inferior to that of the window with the double sash, it was still satisfactory, whereas the watertightness had become markedly worse.

2.1.3 Re-used window with tight inner frames and outer frames with drilled holes

This window was made based on the preceding one by again mounting the additional opening sashes. In order to prevent dew from being formed on the inner surface of the outer glass layer during cold periods, and to ensure balancing of the pressure, two holes with a diameter of seven mm were drilled in the bottom piece of each of the four exterior opening sashes. Thereby ventilation of the air cavity between the sashes was secured. The holes had an outward going slope and were as far from each other as possible, but so that they went free of angle bracings.

At the testing the airtightness was satisfactory. The watertightness was somewhat better than in the preceding case but was, however, still considered unacceptable.

2.2 Window with double-glazing made of recycled wood

The window for this investigation was a traditional two-leafed window, i.e. a double glazed window where the panes are mounted in separate sashes coupled together with a ventilated cavity in between. The window was made of recycled wood. When the windows were received the upper surfaces of the top sashes were totally horizontal so that water could accumulate on them. Besides the joints between main frame and sash were not equally wide in the different parts of the window, e.g. at the left sash, the joint was only about 1 mm wide. This meant that water could "span" over the gap and enter between sash and main frame.

To achieve a satisfactory performance it was necessary to plane the left sash off at the top so it sloped outwards and at the upper part of the sides so that the joint became larger. Besides the wooden sealing list was exchanged with a high elastic sealing profile.

2.3 Conclusion of tests for airtightness and watertightness

The re-used window in all three versions showed after improvement a satisfactory airtightness while in all 3 versions the watertightness was unacceptable after the requirements set up. The window made of recycled wood showed after improvements satisfactory airtightness and watertightness.

Some of the problems attached to the re-used window (the renovated window) are probably due to the fact that the sashes have been turned during renovation. Old sashes often "sagged" and were planed off on the lower part resulting in the overlap in the rabbet being narrower. If the opening frames are turned it gets worse and

besides it might cause the slope of the upper sash to go inward instead of outward.

Corrosion protection of the metal parts is necessary as corrosion was noted after the tests. An acrylic paint is hardly sufficient to prevent the metal from corroding. It is probable that the galvanising of angle bracings and old screws has been damaged by the abrasion during the cleaning process. The new screws are probably only electro-galvanised and it is most likely that damages have occurred at the fixing.

2.4 Comparative durability tests

Two more windows were delivered. They were identical apart from the one being made from new fir, the other from recycled wood. The windows were outward-opening, two-leafed, separated by a mullion and double-glazed. Again the window was with two separate sashes coupled together with a ventilated cavity in between. The outer frame was provided with two horizontal division bars that divided the pane into 3 parts. The purpose of the test was to ascertain signs of changes owing to accelerated ageing and to compare the deterioration between the two windows. One half of the windows was painted with an acrylic based paint system while the other half was left untreated.

In order to follow the moisture conditions, 8 moisture sensors were built into each window, 4 in the painted half and 4 in the untreated one. In each part they were divided with two in the outer frame, one at the top sash and one at the bottom sash, one in the bottom of the main frame and one in the bottom sash of the inner opening light. The moisture content in the sensors was measured as the resistance between two electrodes in the sensor in units of $\log(\text{Mohm})$.

The windows were exposed in the SBI weather-o-meter - "The Four Seasons" - in which the test specimens successively are exposed to ultraviolet radiation and heating (up to approximately 70°C), rain, frost and neutral. 600 cycles were carried out at this test. The moisture measurements were made at the neutral period when the temperature was approximately 20°C.

Very severe moisture conditions have been measured in the tested windows. For windows in practice it is expected that the moisture content over a longer period of time should not exceed 20% if rot and fungus damages are to be avoided. However, the moisture content during the test can not directly be connected to what is experienced in practice as there is no outward water vapour transmission as the method is not simulating interior heating. The risk of rot and fungus damages can consequently not be assessed during the accelerated test described here.

The windows were inspected and photographed 2 and 6 months after exposure was started respectively and again one month after the cease of the exposure of the windows. The results of the ageing test were:

After two months the window putty in the unpainted half had dried out and bursted in some places, especially pronounced at the knots in the division bar. On the painted side of the window the putty has neither burst nor shrunk, but the paint draws the putty away from the window glass so that water can enter between the putty and the window glass. The knots sweat resin so that the paint is discoloured.

For the window made from recycled wood 10 knots three of which were in the division bars could be counted from the outside. For the window made from fresh wood there were 14 knots, but none of them in the division bars.

After six months the putty had dried out considerably since the last registration. The knots in the division bar, see Fig. 2, have burst the bars and is working its way out. There are corrosion formations at the angle bracings and there is water between putty and window glass. In the window made of fresh wood all screws had corroded.

After exposure the main frames on the whole looked satisfying except for a single knot at the hinge which had discoloured the paint and a crack going out from the angle of the main frame.

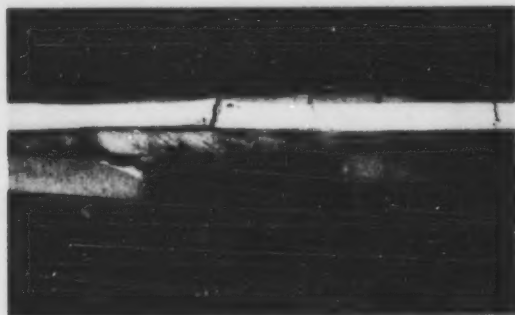


Fig. 2: Knot in a horizontal bar in window made from re-used wood

2.4.3 Conclusion about the comparative durability test

The main frame of the window made from recycled wood had a slightly higher moisture content than the corresponding frame of the new wooden window. However, the difference in performance between the windows tested is so small that it is not possible to draw a conclusion whether there is a difference between the two windows.

If a pre-treatment of the knots with shellac before painting had taken place some of problems connected to deterioration of the paint around the knots could possibly have been avoided. It would also have been better if the grooves had been pre-treated with shellac in order to limit drying-out and cracks in the putty. The painting of the putty with a water-based paint turned out not to be practical when the putty withdrew from the window glass. A painting with linseed oil would probably have been preferable.

To avoid corrosion of the screws they ought to have been either made from stainless steel or to have been hot-galvanised.

The difference in deterioration between the painted and the unpainted part of the windows was not as great as expected. A possible explanation is that once water has penetrated through the paint it can only escape by diffusion which is a very slow process. That could also be the reason why in practice it has turned out that wooden windows of a good quality often do very well without maintenance over decades.

When sorting the wood for making of windows from recycled wood it is important that this is done with care. If the window is to have division bars these should be made of wood of the best quality without knots. For windows made from recycled wood quality wood such as joiner wood and not carpenter wood should be used.

3 Renovation at Øksnehallen - the old slaughterhouse in Copenhagen

Øksnehallen - the old slaughterhouse in Copenhagen - was abandoned as slaughterhouse more than 30 years ago and has since then served different purposes primarily rented to small workshops. The buildings have not been properly maintained from the time they were built at the turn of the century. With financial support from the EU it became possible to make a sustainable renovation of the beautiful old buildings. Included was a renovation of the old windows. As a basis for the decision on how renovation should take place it was decided to make artificial ageing of windows renovated and painted in different ways.

However, the painting on the site could not be delayed by the test and for that reason ageing test and renovation on site took place during the same period.

3.1 Artificial ageing

The purpose of the investigation was to study the effect and durability of different paints and painting procedures in order to assess whether it is possible and feasible to renovate old wooden windows.

The investigation comprised four windows each with four opening lights. Only the opening lights was part of the investigation. The windows were of different wood quality and with different surface treatment according to table 3. The accelerated ageing was performed in the SBI weather-o-meter "The Four Seasons", in the same way as described above for the other project. The acceleration factor is from earlier experiments with similar materials known to be roughly 10 x natural ageing (C. Woetmann Nielsen, 1995).

Table 1: Wood and surface coatings used for the tests at Øksnehallen

	a-sashes	b-sashes	c-sashes	d-sashes
Window 1	Old wood medium quality <i>Linseed paint</i>	Old wood medium quality <i>Wood preserv. + linseed paint</i>	Old wood medium quality <i>Vac. Impreg. + waterbased paint</i>	Old wood medium quality <i>Wood preser. + Waterbased paint</i>
Window 2	Old wood good quality <i>Linseed paint</i>	Old wood Good quality <i>Wood preserv. + linseed paint</i>	Old wood Good quality <i>Vac. Impreg. + waterbased paint</i>	Old wood good quality <i>Wood preser. + Waterbased paint</i>
Window 3	New spruce <i>Linseed paint</i>	New spruce <i>Wood preserv. + linseed paint</i>	New spruce <i>Vac. Impreg. + waterbased paint</i>	New spruce <i>Wood preser. + Waterbased paint</i>
Window 4	New fir <i>Linseed paint</i>	New fir <i>Wood preserv. + linseed paint</i>	New fir <i>Vac. Impreg. + waterbased paint</i>	New fir <i>Wood preser. + Waterbased paint</i>

During the ageing the moisture content was measured continuously, i.e. measuring every week for the 26 weeks the artificial ageing lasted. The results of the test were assessed as the performance of the sashes as regards water uptake and visual changes e.g. cracks in the wood.

The results can be summarised as:

- The moisture uptake was less in new than in old wood
- The moisture uptake was less in wood of good quality (whether new or old) than in wood of inferior quality. It should be noticed that the quality of the old wood, even for the wood classified as good, not was as high as expected.
- The moisture uptake was less in vacuum impregnated wood painted with water-based paint than in wood painted with linseed oil based paint
- There were no visible difference on whether the wood had been treated with wood preservatives or not. This might be due to the fact that there is no possibility of germination during the artificial ageing.

3.2 In situ investigation

When the project began the windows in the studied building had the typical failures – the lower part of some sashes and sometimes also the sides had attacks of fungus and dry rot. Especially the condition under the angle bracings and at screw holes was often bad. In general the wood was of a good quality and the overall condition of the windows turned out to be better than at first sight. For this reason it was decided not to change the windows but to renovate them.

All windows in the building were completely removed for renovation but marked so that each one came back to its original position. The sashes were cleaned, i.e. all old paint was removed by mechanical scraping combined with softening with IR lamps (which do not harm the glass). Afterwards they were repaired by carpenters

who in some cases only made minor repairs in other cases they replaced the most deteriorated part, more often the lower sash. In this way it was possible to re-use about 90% of the windows, of which 85-90% only had minor repairs.

After being repaired the windows were saturated with cold pressed linseed oil. The oil was allowed to dry sufficiently before finally the windows were painted twice with linseed-oil based paint. Rabbets and knots were sealed with shellac before the painting, in order to avoid drying out of the putty due to the oil migrating into the wood. The linseed oil based paint was manufactured in a small factory.

Finally the windows were placed in their original positions in the building. Due to the repairs, changes of some angle bracings, handling on site etc. some minor repair works had to be done before the building was handed over to the owner – the municipality of Copenhagen.

The windows were surveyed in the summer 1998 about 3 years after the repair/painting with the following results:

- The moisture content measured with a hand held moisture meter was generally 7-9% only in a few cases moisture contents of 13-15% were registered, and these were all near surface cracks.
- All repairs seems to have performed well as there were no signs of deterioration or flaking around the repairs which were done with ordinary PVA glue, see Fig. 3.
- The attachment of the paint to the surface was excellent.
- The appearance was unsatisfactory as the paint appeared mat and almost black instead of the original more glossy green-black. However, the surface is assessed to be acceptable. Partially the paint was much lighter and greener than the rest. This appears to be the repairs of the windows after they were remounted in the building. Apparently this problem is caused by the manufacturer not having sufficient quality control with the composition of the paint.



Fig. 3: Repair in windows after three years of natural exposure

4 Conclusions and comments

The preconditions of the ageing tests and the in situ tests have been quite different and the results can for that reason not be directly compared. On the other hand they can supplement each other and to some extent confirm each other. On basis of the investigations it can be concluded:

- Windows to be re-used after a renovation must be selected and sorted with great care according to whether they are to be re-used totally, partly or not at all.
- At renovation of opening frames normal requirements to design/geometrical parameters must be fulfilled, e.g. outwards going slope on surfaces facing upwards and sufficient joints between outer frame and opening lights must be secured.
- Corrosion protection should be carried out when necessary, e.g. when abrasion of metal has taken place and when new screws have been inserted. Alternatively, the angle bracings can be replaced and fixed with sufficiently corrosion-protected screws, e.g. warm-galvanised, or stainless steel.
- The quality of the wood has a decisive influence on the result. Good quality will, all other factors the same, give the best result. It is not possible to compensate for deteriorated wood with a surface treatment/painting in order to achieve a good result. Knots should be avoided as far as possible especially in profiles with small dimensions.
- The workmanship including the necessary time to allow drying after linseed treatment is crucial to good saturation of the wood and good "sealing" of the surface. If sufficient care is not taken to the working process linseed oil based painting seems not to be as protective as traditional paint systems in current use.
- Rabbets, knots etc. should be pre-treated with shellac to avoid drying out of the putty.
- Repair works should be finished before painting as any penetrations of the paint coat are potential gateways to water.
- Linseed oil based paints are not in itself suitable for protection of metal-works like angle bracings etc.

The in situ investigations on the old slaughterhouse showed that:

- In practice the linseed oil based paint gives a good protection of the surfaces against water as no dangerous moisture content was recorded at the survey, even though measurements were performed after several days of rain.
- There are problems regarding composition of the linseed oil products. Partly these problems are due to insufficient quality assurance which means that the quality is not identical from one time to another. Partly the problems are due to insufficient properties as regards for instance application of the paint, i.e. it is not easy to work with, and long-term properties including chalking of the surface after exposure. The latter also gives a contribution to the

uncertainty of the artificial ageing tests as in reality it is not known whether the products tested had the same properties as those from the in situ investigations.

- A good result is dependent on a sufficient period in between applications, i.e. thin layers and reasonable temperature- and humidity conditions.

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63 MEASURING MOISTURE CONTENT IN WOOD WITH BUILT IN PROBES — 20+ YEARS EXPERIENCE

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Abstract

In Denmark probes made with electrodes placed in a small plywood disc or a small piece of beech wood have been used for long-term measurement of moisture content in wooden constructions, especially roofs. The probes are embedded in the wood structure where the measurements are to be made and are equipped with wires in order to make remote measurements. This makes it possible to measure moisture content in closed cavities etc, where it would be difficult or impossible to use hand-held moisture meters. The method is described in NORDTEST method 420 "Building materials, wood: Moisture content".

The paper describes the experiences gathered during the many years of use. A large amount of constructions have been equipped with this type of sensors giving experience about the usefulness and the long-term stability of the sensors. Besides, different investigations have taken place regarding, for instance, the influence of the shape and dimensions of the probes, the materials used for electrodes and the way of connecting the wires to the electrodes. For some years it has been feasible to perform "continuous" measurements with a datalogger. An example of results achieved in this way is given.

The precision, the advantages and drawbacks experienced during the use etc. is discussed. Comparisons are made to what can be achieved by placing electrodes directly in the wood to be measured.

Keywords: Durability, humidity, moisture content, moisture sensors, test method.

1 Introduction

In Denmark the same principle as the one used in the hand-held moisture meters was applied during extensive roof surveys in the early 1970's, when the so-called moisture measuring discs were developed. The moisture measuring discs have since then been used quite intensively both for research work and for practical purposes where there was a wish of monitoring the moisture content of wooden constructions. The use of the moisture measuring discs are described e.g. in (Prebensen and Brandt 1984 and PHØNIX Posten, 1984) whereas the moisture measuring dowels are described in (Munch-Petersen, 1990). Finally the method has been described in detail in (Nordtest Method NT Build 420, 1993). The experience with the method is good but in measurements carried out at the Danish Building Research Institute, (SBI), it proved difficult to obtain concurrent results with moisture probes manufactured in different ways. Consequently a series of investigations were carried out to reveal the influence of different factors on the measurements. These results are examined and commented upon in the following. Results from moisture measuring disc and moisture measuring dowel as well as for electrodes placed directly in the wood to be measured are described.

2 The advantage of built-in probes

The advantage of built in probes is that they are inserted permanently which means they are able to monitor the moisture conditions over a period of time. In this way it is possible to monitor variations in moisture content e.g. due to seasonal variations or changes in use of the construction. Another asset is that it is possible to detect any abnormal changes in the behaviour of a construction, e.g. it is possible to follow the moisture content in new innovative constructions to ensure that the moisture content remains on the expected (low) level.

Finally, it is an advantage that the electrodes can be built in right from the beginning so that measurements can be carried out in inaccessible places, for instance in cavities. For this purpose it is only necessary to wire the sensors to a place convenient to measurement and registration.

The advantage of using an intermediary link in the form of moisture measuring disc or moisture measuring dowel is that it assures that measuring is taking place in relatively homogeneous material with well known properties and that a direct relation exists between the measured resistance and the moisture content. Practical experience also shows that more stable measurements are obtained. The disadvantage of using probes is that it is necessary to calibrate and that, consequently, the price is relatively high.

3 Description and use of probes

The moisture measuring disc consists of a 12 mm plywood disc with a diameter of 50 mm. Originally the plywood was of the same type as the one in which the moisture content was to be measured. Two electrodes are inserted in the plywood. Besides, the disc is equipped with a thermocouple, which is used for temperature registration as well as for temperature compensation of the resistance. Before use the moisture measuring disc's are calibrated by being placed in turn under different, controlled humidity conditions. In this way it is possible to find a correlation between the resistance and the moisture content of the plywood. At calibration, probes differing from the rest are identified and discarded.

At a later stage moisture measuring dowel's were developed. These are in principle similar to the discs only differing by being made of beech wood and in somewhat smaller dimensions.



Fig. 1: Plywood moisture measuring disc 50 mm Ø and beech wood dowel 10 mm Ø

3.1 The use of moisture measuring disc and moisture measuring dowel

Moisture measuring discs and moisture measuring dowel have been available for quite a few years and the method has now been made into a NORDTEST method: NT BUILD 420 "Building materials; Wood; Moisture content".

After calibration the probes are embedded in the wood or plywood under investigation. After moisture equilibrium with the surroundings has been achieved the resistance between the electrodes is measured and thereby, indirectly, it is possible to find the moisture content in the probes and the surrounding wooden material. Measuring by hand is usually done by means of a log(Mohm)-meter but the electrodes may also be hooked up to an ordinary hand held moisture meter in which case they function as an extension of the legs/pins of the meter, see Fig. 2.



Fig. 2: Hand-held moisture meters and log(Mohm)-meter

Until a few years ago it was rather cumbersome and expensive to collect data with a datalogger as the loggers were unsuitable for the purpose. There are, however, now loggers on the market which function well with moisture measuring disc and moisture measuring dowel. Suitable loggers should have a great input resistance because of the large resistance between electrodes in wood. Besides it shall use AC or alternating DC in order to avoid polarisation of the fluid in the pores of the wood. At temperatures other than 20EC, correction must be made. For example a deviation of - 8EC (i.e. measurement at 12EC) means a moisture reading which is approximately one per cent too low. The temperature correction is described in NT-Build 420.

4 Design parameters

Several tests have been performed as regards the influence of the mounting method of the wires to the electrodes as well as the choice of electrode material and the size and shape of the probes. The results of these tests are described below.

4.1 Mounting of wire

To find the influence of the electrode material a test was performed where brass nails, copper nails, stripped wire and Ø 1.5 mm cable wire were used as electrodes. The mounting of the wires were made by mechanical contact to the electrodes and a soldered solution respectively.

The test results showed that all readings followed very closely the actual moisture content of the wood which was followed also by weighing the specimens during the tests, see Fig. 3. No significant difference was found between the various electrode materials.

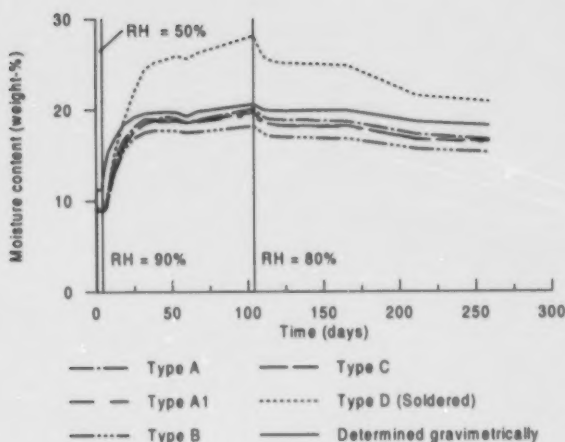


Fig. 3: Moisture content in various moisture measuring dowels subjected to varying relative humidity as a function of time

As regards the effect of the mounting of the wires to the electrodes it was found that soldering of the wires instead of mechanical fastening gives less resistance between the electrodes. Apparently soldering of the wire to electrodes already fixed in the wood causes some permanent changes in the resistance between the electrodes. This is probably due to heating of the wood during the soldering. Consequently, different calibration curves shall be used depending on the fixing of the wires. Calibration of probes should therefore not take place until after the wires have been fixed to the electrodes so that the effect of the fixing process is taken into account in the calibration.

4.2 Types of electrodes

For hand-held moisture meters it is well known that electrodes may either be insulated or bare (non-insulated), see Fig. 4.

For the insulated electrodes only the tip of the electrode is bare and the resistance is consequently measured between the two tips of the electrodes. If the electrodes are plugged into the wood it is, therefore, the moisture content of the wood that is decisive for the result of a measurement - as it should be.

Contrary for bare electrodes, the resistance can be measured between any points of the electrodes. The resistance registered is the lowest between any two points of the electrodes. If, for instance, condensation exists on a surface, it is the resistance in the condensation-layer that is the lowest and therefore the condensation becomes decisive for the result of the measurement. The result of the measurement is therefore unlikely to be representative of the moisture content inside the wood.

For use in probes, ready-made electrodes in the form of surface coated (insulated) nails are available.

If the probes are intended for use in places where so high humidity levels are

expected, that there is a risk of corrosion, electrodes of corrosion resistant materials should be used.

This can, for instance be done by drilling a narrow hole equal to the diameter of a thin plastic hose. A piece of hose is inserted into the hole and a nail, for example of stainless steel, is led through the hose and hammered into the wood so only the tip penetrates the hose. The hose functions as an insulator against the surroundings.

For hand held meters, bare electrodes are the most common ones on the market due to price. As regards fixed electrodes, the price difference between the use of bare and insulated electrodes is so minimal that there is no reason to use bare electrodes.

Recently the Danish Building Research Institute has started to use gold plated precision manufactured contacts, meant for use in connectors, as electrodes in moisture measuring discs. The wires are fastened to the contacts by means of a crimp-tool.

4.3 Size and shape of probe

With a look to the difference in size of dowels and discs it seemed worthwhile to test the influence of shape and size of the probes, as this could open up for the use of probes "tailor-made" to specific purposes.

From previous tests it is well known that distance between the electrodes has only a minor influence on the resistance (correct within wide limits). Therefore experiments have been made regarding the influence of the design of probes using different sizes, shapes and electrode distances and designs.

The results showed that design is of no influence. The probe can, according to need, be made in the size and shape suitable for the task in question.

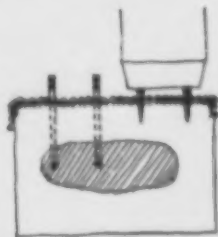


Fig. 4: Measurement of moisture content (resistance) with bare and insulated electrodes respectively. With bare electrodes the lowest resistance is measured. In case of surface condensation, the condensation is decisive for the result and not the moisture content inside the wood

5 Experience from the use of moisture measuring probes

Long-term experience with regard to discs and dowels for moisture measurement has been favorable. The long-term stability has been examined through measurements made on 20+ year old discs in a roof construction. These measurements show that the long-term stability is good.

A more inexpensive solution is to place electrodes directly in the wooden construction to be examined. The mounting is simple and inexpensive and if the only purpose is to observe changes in the moisture content and not the absolute moisture content, electrodes plugged directly into the wood in the construction could prove beneficial.

By use of electrodes plugged directly into the wood it has proved practical to apply ordinary terminal blocks for measuring. Measurement can be carried out either directly on a coupler of conduits attached to the electrodes, or wires can be led from the electrodes to a suitable measuring point where they end in a terminal block. If the two outer links in a three-light terminal block are used the pins of an ordinary hand-held moisture meter can be used for a direct measurement of the moisture content in the desired depth, see Fig. 5.



Fig. 5: Measurement of moisture content in various depths using a hand-held moisture meter and three-light terminal blocks

6 Examples of usage of built-in electrodes for moisture measurement

Over the years built-in electrodes have been applied both for laboratory tests and in connection with long-term surveillance of renovation solutions or new types of construction.

As an example of usage for laboratory purposes can be mentioned usage in connection with surveys of roof constructions and windows.

6.1 Moisture conditions in unventilated roofs

In connection with tests regarding moisture conditions in unventilated roof tile underlays in the SBI moisture test house, moisture was measured in the upper and lower surface of rafters as well as in counter battens. For this purpose a large amount of dowels have been used which were built into the elements before erection. The wires were hooked up to a datalogger inside the house. A regular registration of the moisture conditions in the various roof elements was performed every 6 hours for 2 years. This was done in order to monitor the moisture content in the wooden construction thereby making it possible to judge whether there was a risk of

degradation or not.

6.2 Accelerated ageing of windows

When performing accelerated ageing tests on wooden windows, insulated electrodes were inserted directly into the frames in order to register the moisture accumulation and the drying-out, all depending on the position in the window. In these cases the electrodes were inserted where the greatest risk of moisture pick-up was likely to occur, for example 5 mm from the outer surface of the frame or 5 mm below the edge of the window fillet/groove, see sketch on Fig. 6. The electrodes were inserted from the backside of the window (the inner surface of the window) and the wires were hooked up to a measuring panel of terminal blocks fixed to a place convenient for manual measurement. Measurement took place with a hand-held moisture meter once a week. Reading of approximately 30 measurements could be done in less than 30 minutes.

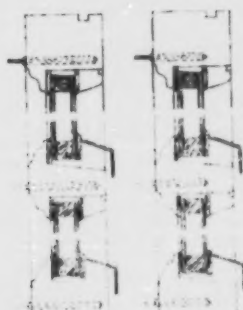


Fig. 6: Position of electrodes in window frames during accelerated ageing test

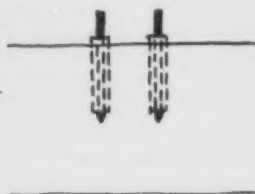


Fig. 7: Insulation of electrodes with plastic hose

6.3 Tests concerning absorption and drying-out

In the case of pilot projects or damage investigations, there is often a need for measuring the moisture content in certain depths to ascertain the moisture distribution in the construction. In order to illustrate these possibilities, some tests with a series of insulated electrodes embedded in different depths were made. The electrodes are executed as described earlier with a drilled hole into which a thin plastic hose is inserted. Through this hose, are inserted electrodes of brass or copper so only the tip protrudes, see Fig. 7. Identical tests with fir sapwood, fir hearth-wood and spruce were carried out.

The electrodes are placed in heights varying from 5 mm to 40 mm above the bottom of the wood specimen. As regards the first pair of electrodes they are placed approximately 15 mm from the end of the wood specimen, and as to the last pair of electrodes approximately 120 mm from the opposite end. The electrodes are placed in

rows, for fir two rows of sapwood and two of heartwood respectively. Half of the measuring points are doubled so that some absorption of the moisture through the end grain specimen can take place at one end of the wood specimen while at the other end a wetting/spraying primarily takes place through the side wood, see Fig. 8. The wood specimens were placed with the bottoms emerged 3-5 mm in a water bath for approximately 30 days. The moisture content is measured regularly by connecting the electrodes to an ordinary hand-held moisture meter by means of a terminal block. Close to the end of the specimen a rather quick wetting took place especially in the three pairs of electrodes in the deepest position (closest to the end grain). Also the remaining pairs of electrodes showed a faster wetting due to the influence of the end-grain wood (absorption). Very distinct differences in the wetting and drying-out cycles for the various qualities can be observed as the sapwood is wetted much faster and more thoroughly than heartwood which again is wetted to a higher degree than spruce. The test clearly shows that insulated electrodes are a suitable way of determining the moisture content in different depths.

Subsequently approximately 4 weeks of drying-out took place by means of heat lamps placed below the wood pieces (so that the moist bottom was heated up). From Fig. 9 it can be observed how the heating causes an instant rise in the moisture content because the temperature drives the moisture into the wood by diffusion. This corresponds to conditions under which a moist window frame is exposed to sunlight

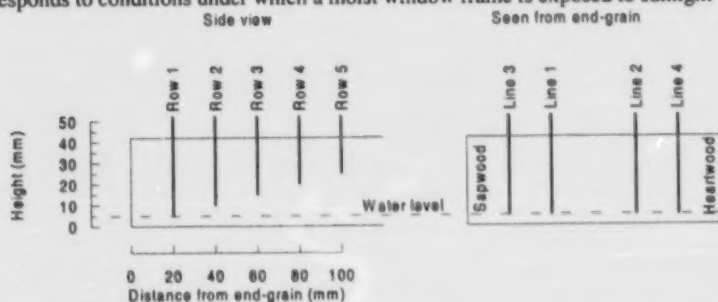


Fig. 8: Position of electrodes in wood specimen during wetting and drying experiment

where the heating will drive the moisture into the wood. The temperature and thereby the drive pressure will increase on dark surfaces. This fact is confirmed by investigations concerning moisture measurement by means of electrodes in window frames and beams in the SBI moisture test house.

For the highest-lying pairs of electrodes a certain delay can be observed both during wetting and during the following drying-out. This corresponds to the general experience that moisture accumulation and drying-out processes are dependent on the dimensions of the wooden pieces. Drying-out by diffusion through the wood is a slow process and is even slower when there are only small variations in the vapour pressure

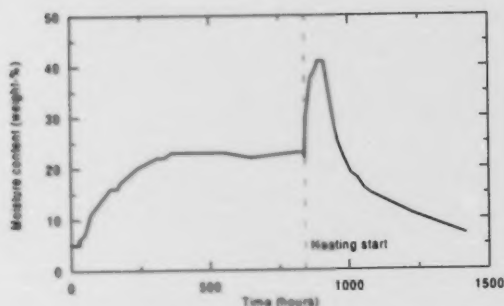


Fig. 9: Moisture content in wood specimen during water uptake and subsequent heating

7 Usage in practice

Usage in practice is mainly to be found in roof constructions. Normally dowels or discs are installed in connection with erection/renovation and wires for measurement purposes are led to a convenient measuring point. When it is a question of manual reading it will, as a rule, only be possible to make one or at best a few readings annually. As far as possible the measurements should be made at the same time of the year each time so that the influence of the seasonal variation in the moisture conditions does not become too great. When assessing the measurement results, consideration should be given to how the moisture variations could behave outside the moment of measuring.

In components exposed to the environment (roofs, outer walls, etc.) it would normally be most suitable to make measurements during the period of February-March (in the northern hemisphere) at which time the highest moisture content is to be found in the constructions.

8 Conclusion

Based on experience moisture measurement in wood by means of discs, dowels or built-in electrodes must be considered as a good and reliable method. With insulated electrodes (nails), it is possible to apply the method for measuring in certain depths, for instance when assessing the moisture distribution. Measurement can be made by means of an ordinary contact plug meter and a 3-light terminal block. Experience shows that the long-term stability of built-in sensors/electrodes is good.

Furthermore it is important to remember the following before a moisture measuring is initiated:

- Knowledge of building physics is a prerequisite when interpreting the results exceeding ordinary moisture control.
- The moisture conditions in a construction may vary considerably from one area to another and consequently several measurements are necessary to give a clear picture of the conditions.
- The moisture conditions may vary considerably over the year and an assessment of the results should take this fact into consideration.
- As with hand-held moisture meters measurements should be made along the grain. It should be avoided to measure near cracks or nails or in impregnated wood.

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64 IN-SERVICE MOISTURE CONTENT OF HARDBOARD LAP SIDING IN SOUTHERN FLORIDA

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Abstract

To evaluate the effect of backpriming on in-service performance, hardboard lap siding from one manufacturing plant was exposed on two test buildings in southern Florida for 29 months. The two buildings were identical, except that one had 0.3 m (12-inch) roof overhangs without gutters and the other had gutters but no roof overhangs. Siding installation was the same on both buildings and was essentially as recommended by the manufacturer. Laboratory tests revealed that edge swell and other properties of this particular siding were much better than required by industry standards. With only one exception, noticeable caulk seal failures either did not occur or were promptly corrected. All siding remained dry and in excellent condition over the exposure period. Some water intrusion occurred between some of the window units and trim. The siding system dissipated this water intrusion without suffering damage. Roof overhangs reduced moisture levels in the exterior millwork and trim, but did not appear to reduce the moisture content of this siding. Backpriming did not improve the performance of this particular hardboard siding nor did it lower in-service moisture content.

Keywords: hardboard siding, moisture content, backpriming, effect of roof overhangs, effect of gutters.

1 Introduction

In response to incidences of moisture-induced deterioration of hardboard siding in south Florida, the U.S. Department of Housing and Urban Development (HUD) proposed a Local Acceptable Standard (LAS) for southern Florida, which prescribed factory prefinishing of the siding and priming all surfaces (including the back surface). The field study reported in this paper was intended to address the

uncertainty about the necessity and effectiveness of these proposed measures. The study's objective was to determine if backpriming or factory pre-finishing might improve the durability of hardboard lap siding when installed according to recommended practice.

To determine the effects of backpriming and factory pre-finishing on the potential for deterioration, we monitored moisture content, thickness swell and appearance of site-finished, factory finished, backprimed, and non-backprimed hardboard siding on two full-size buildings in Florida. Although we expected that two years of exposure would not induce siding failure, we envisioned that differences in moisture content, thickness swell, or appearance would indicate potential differences in performance or durability. This paper presents some study findings; it is limited to observations of moisture history and physical condition of siding and of exterior wood millwork and trim.

2 Methods

The two test buildings were in Palm Beach County near the Atlantic coast of southern Florida. Siding installation was the same on both buildings and was essentially as recommended by the manufacturer. The siding on two test buildings was finished with four different finish treatments. The siding was on the building for approximately 29 months, after which the buildings were demolished. Temperature and humidity inside the buildings were controlled. We continuously monitored the moisture content and periodically visually inspected the siding.

2.1 Test buildings

One building had 0.3-m (12-inch) overhangs, including the gable ends, and had no rain gutters; the other building had no roof overhangs, but had rain gutters on the eaves. The buildings were otherwise identical. Dimensions of the buildings were 9.75 by 9.75 m (32 by 32 ft). The gable ends faced northeast and southwest. Walls were wood frame nominal 2x4 construction and were insulated with glass fiber batt insulation. The interior of the walls and ceilings was finished with taped and painted gypsum wallboard. Each building was air-conditioned, and distribution ducts were within the conditioned interior (not in the attic), while ceiling fans provided additional air mixing.

Each side of the buildings consisted of four 2.4 m (8 ft) long wall sections of varying construction; most were sheathed with plywood, while some had no sheathing. All walls included a secondary weather barrier, either #15 asphalted felt or woven polyolefin sheet. To prevent air movement between them, adjacent wall sections were separated with pressure-treated nominal 2x6 lumber, sandwiched between the end studs of the wall sections, except at outside corners. The outside edge of the 2x6 separator protruded beyond the face of the siding. At outside corners, walls were joined in a standard manner, with vertical outside corner trim consisting of western redcedar (*Thuja plicata*) strips.

Two doors and 12 windows were installed in each building. The doors were steel-door units with pine jambs. The windows were aluminum single-hung units with side nailing flanges. Exterior trim for windows and doors was site-fabricated from western redcedar strips. Window trim was installed over the window flanges. The

piece of trim at the bottom of the window was sloped to the exterior. The trim was caulked to the window unit with urethane caulk, and joints between the pieces of trim were also caulked. Aluminum head flashing was installed over the top cedar trim piece above all window and door units.

The buildings were not shaded by trees, and there was no shrubbery within 6 m (20 ft) of either building. Gutters on the building without overhangs remained free of blockage over the course of the study.

2.2 Siding

The siding was 203-mm (8-inch) wide, 10-mm (0.2-inch) thick hardboard lap siding from one mill. We performed weatherability of substrate tests and linear expansion tests, as described in Standard AHA/ANSI A135 (American Hardboard Association 1990), on samples of this siding. The weatherability of substrate test provides a measure of the board's permanent thickness swell when subjected to a series of wetting and drying cycles. The linear expansion test measures the board's change in length when brought to equilibrium at 90% relative humidity (RH) from equilibrium at 30% RH. The data (Table 1) indicate that this siding's properties substantially exceeded the minimum values required in the industry standard.

Table 1: Comparison of test results on boards used in this study and industry minimum standards

	Residual Swell ¹ , average and range (%)	Linear expansion ² , average and range (%)
Study material	4.3 (1.6-7.4)	0.23 (0.21-0.27)
ANSI/AHA requirement ³	20 maximum	0.40 maximum

¹ Weatherability of Substrate test as per section 4.2 of ANSI/AHA standard.

² 30% to 90% RH, in machine direction, as per Table 2 of ANSI/AHA standard.

³ ANSI/AHA A1356-1990, American National Standard for Hardboard Siding

Table 2: Equilibrium moisture content data

RH (%)	EMC of hardboard siding ¹ (%)	EMC of southern pine ² (%)	EMC of construction plywood ² (%)
30	3.9	5.3	4.8
43	4.7	7.0	6.3
65	6.8	10.8	9.7
79	8.9	14.8	13.4
90	10.4	20.0	18.5

¹ sorption only

² Source: Richards et al. (1992)

We measured equilibrium moisture content (EMC) of the hardboard siding (Table 2). The siding has lower EMC than solid wood or plywood at corresponding relative humidity conditions. Values for hardboard in Table 2 are in general agreement with previously published values (Bristow and Back, 1969) for heat-

treated hardboards.

2.3 Siding finishes

The exterior surface of all siding was either factory-primed or factory-primed and factory-finished. Factory-primed board was topcoated with a site-applied acrylic paint after installation on the buildings. About two months prior to installation, we primed the back surface of roughly half the boards with an alkyd ("oil-based") primer. Backs of the rest of the boards were left un-primed, although all siding had a thin and incomplete cover of primer or finish coating on the back as it was received from the mill¹. This cover of primer or paint on the backs was generally continuous at the drip edges, although thin.

Finish treatments were: 1) backprimed, factory finished [BF]; 2) not backprimed, factory finished [NF]; 3) backprimed, one coat of site-applied finish [BS]; 4) not backprimed, one coat of site-applied finish [NS]. Site-applied finish was applied within ten days after siding installation. We used an acrylic-latex gloss exterior paint, applied by brush by a professional painter. The paint color was similar to that of the factory finished siding. Good coverage was obtained, and a mirror was used to inspect for full coverage of the drip edge.

2.4 Siding and trim installation

The siding was installed during the last week of November and first two weeks of December 1994. Siding moisture content at installation ranged from 5% to 7%, measured gravimetrically. The siding was manually face-nailed with galvanized nails. The minimum overlap between boards was 25 mm (1 inch). Siding crosscuts were left unpainted. Window, door, and outside corner trim were installed before the siding, as were the 2x6 wall section separators. All joints between siding and either trim or wall separators were caulked with urethane caulk. Gable end cuts of the siding on the building with overhangs were covered by aluminum soffit. On the building with overhangs they were covered with redcedar gable end trim, the top edge of which was in turn covered by metal roof edge. Gable end cuts were therefore protected from the weather, and did not rely on caulk seals for that protection.

The four different combinations of siding finish treatments were alternated vertically, according to the sequence (going upward): BS, NS, BF, NF. The finish treatment for the bottom siding course varied from wall section to wall section, such that all finish treatments were equally allocated to bottom-course locations.

2.5 Interior and exterior conditions

Between February 1995 and study termination, interior conditions in the buildings were maintained at approximately 75 °F and 50% relative humidity (RH).

On-site rain gauge data were manually collected from April of 1995 until study termination, generally on weekly intervals. Data from the nearest National Weather Service (NWS) station at West Palm Beach, about 16 miles away, were in close agreement with site-collected data. Annual rainfall at the site was approximately 59 inches per year, close to the historical average (years 1961-1990) of 61 inches per

¹ Paint on siding backs was incidental to the factory priming and/or painting of front surfaces and drip edges.

year at the NWS site in West Palm Beach.

We used the NWS data for April 1995 through April 1997 to calculate the climatic decay hazard index (Scheffer, 1971). That index was developed as an indicator of the potential for decay of wood on building exteriors above ground. The calculated value for the exposure period was 154. Index values in excess of 65, based on historic average data, indicate climates of high decay hazard. Index values for the continental U.S., based on historic average data, exceed 110 only for the Florida peninsula (Scheffer, 1971).

2.6 Instrumentation and automated monitoring

On each building 46 siding boards were instrumented with thermocouples and moisture content pins. They were placed in a variety of exposures. Placement was essentially identical on both buildings. Of the 46 instrumented boards, 12 were in bottom-course positions, 28 abutted a window or door, and 6 were installed high on the walls (approximately at ceiling height). The four finish combinations were equally represented on each side of the building, and by type of exposure.

Moisture content was determined from direct current (DC) resistance between pins and adjusted for board temperature. We calibrated the pins for this hardboard siding at room temperature, and corrected for temperature as suggested by James (1975). The insulated MC pins were installed on the front faces of siding boards in pre-drilled holes and carefully caulked to prevent water entry into the holes. After attachment of the leads, the pins were covered with caulk for protection from rain. Leads were connected to signal conditioners with circuitry as shown in Carll and TenWolde (1996). Type T thermocouples were embedded in the boards, through holes drilled from the back of the board to a depth of about 5 mm (0.2 inch). Each building had its own data collection system, controlled by a personal computer. Readings on instrumented boards were taken hourly. Data collection commenced in April 1995 and ended on May 6, 1997.

2.7 Data reliability

The signal conditioner circuits for the moisture pins could not measure resistance exceeding the equivalent to between-pin resistance of the hardboard siding at room temperature and 7% MC. Therefore, with siding at room temperature, 7% MC was approximately the lowest measurable value. However, because the electrical resistance of wood at a given moisture content decreases with increases in temperature, we were able to measure down to approximately 6% MC when the siding boards were heated by the sun.

We used insulated pins so that wet board surfaces would not result in erroneous high moisture content readings. However, abrupt changes in MC readings sometimes occurred, and were most likely related to surface wetting, not to an actual increase in the MC of the siding. Average daily readings for any group of pin pairs that were above 9% MC appeared to be associated with abrupt jumps in individual pin pair readings, and were thus suspect.

Prior to February 10, 1996, there were errors in our MC measurements due to instrumentation and calibration problems. After correcting these, moisture pin readings made by the data collection system were compared with moisture pin readings from a DC resistance moisture meter, with siding temperature close to meter calibration temperature. The values agreed within 1% MC. At study completion,

samples of siding were removed from the buildings within 3 days of termination of automated data collection and their moisture contents determined gravimetrically. Average moisture pin readings for the final week of automated data collection, grouped by building and wall type, were within 1.5% MC of the corresponding final in-service MC's determined gravimetrically.

2.8 On-site inspections

2.8.1 Quarterly inspections

A total of ten quarterly inspections were performed. The first inspection was shortly after installation and painting of the siding in December 1994, and the final inspection was shortly prior to siding removal in May 1997. At each inspection, the appearance of each piece of siding was noted and thickness of the lower left and lower right edges of each siding board was measured in place using a specially designed tool incorporating a dial gauge.

2.8.2 Final observations and measurements

On May 5 and 6, 1997 we measured the moisture content of wood trim on each building (with the exception of the fascia behind the gutters of the building without overhangs), with a DC resistance moisture meter. On May 6-9, 1997 we removed siding from the buildings, except on gable ends above ceiling height, inspected siding back surfaces, and looked for evidence of rainwater leakage. On each building we cut a total of 128 siding specimens, representing all 32 wall sections, for gravimetric determination of moisture content. Five additional moisture content specimens were cut directly underneath windows on the building with gutters; these were windows which showed evidence of rainwater leakage.

3 Results

3.1 Quarterly site inspections

Throughout the exposure period, the siding remained in excellent condition. No edge puckering was observed on any board on either building, nor was siding buckling or finish failure. Furthermore, we never observed any cracking of paint on drip edges. This indicates that the siding remained dry over the exposure period. In some shaded locations however, mildew growth occurred on board surfaces. Noticeable caulk failures between siding and trim either did not occur, or occurred early in the exposure period and were repaired. The exception was one caulk failure between a window and its trim unit that was noticed after 9 months of exposure and was left uncorrected for the remaining 20 months. Siding backs below this window were stained when we removed the siding in May 1997, indicating that water intrusion did occur.

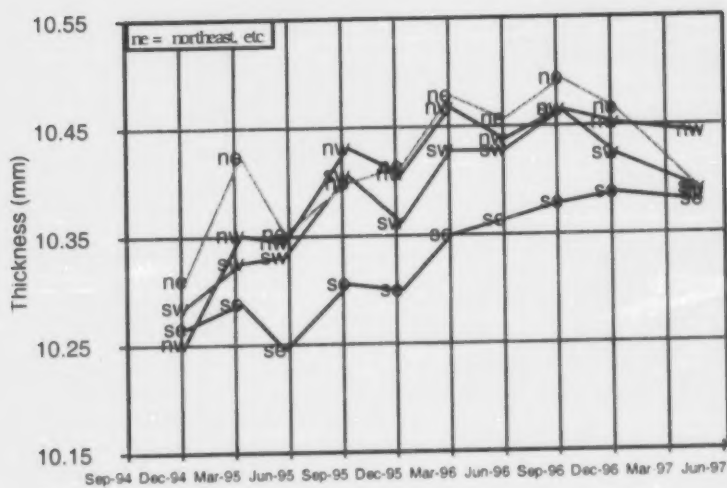


Fig. 1(a): Results of thickness measurements on building with gutters, by wall orientation

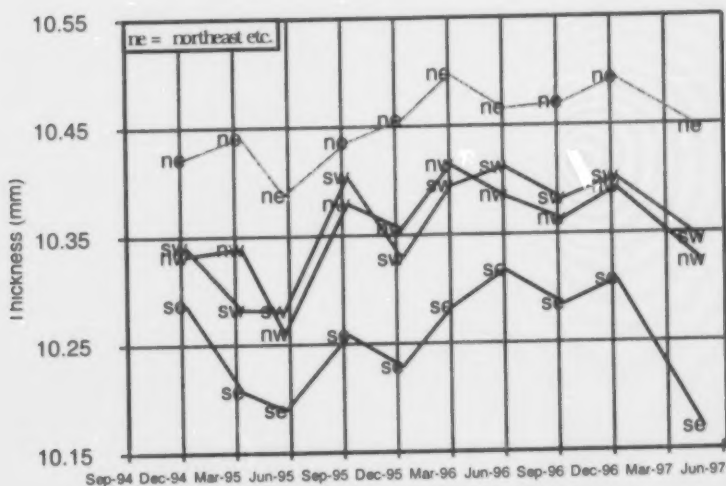


Fig. 1(b): Results of thickness measurements on building with overhangs, by wall orientation

Figures 1A and 1B show the results of the in-place thickness measurements. Thickness increased very little after March 1995 (3 months of exposure) and after March 1996 (15 months of exposure) no long-term increase is apparent. Based on an estimated initial thickness of 10 mm (0.4 inch), thickness swell stayed below 3% over the entire exposure period, while the average thickness swell was about 2%. Figures 1A and 1B also show a seasonal fluctuation in thickness, with some shrinkage during summer and swell during winter. Generally, siding on the shady sides of the buildings was thicker than boards on the sunny sides. Neither finish system, nor building (building with gutters versus building with overhangs) showed any consistent significant effect on in-place thickness.

3.2 Siding moisture contents

3.2.1 Moisture contents during exposure

Moisture pin data indicated that the siding remained below 9% MC over the monitoring period. Figure 2 shows daily average MC for all moisture pin readings on each building from February 10, 1996 until termination of data collection on May 6, 1997. Data collected prior to February 10, 1996 are not presented because of the calibration and instrumentation problems mentioned previously. Figure 2 shows a few short periods with a daily average MC above 9%. These corresponded with rain events, and, as indicated previously, are probably erroneous. The seasonal fluctuation in MC level shows a similar trend as fluctuation in in-place thickness data, i.e. lower MC in summer, higher MC during winter.

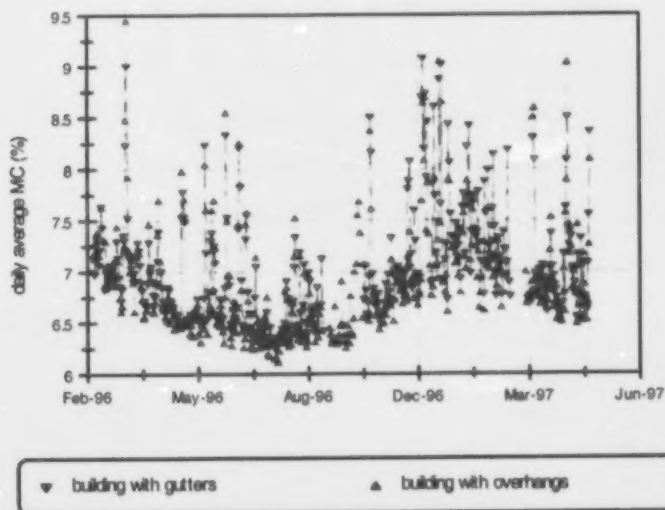


Fig. 2: Daily average moisture content readings from moisture pins, from February 10, 1996 through study completion

The data show no discernable differences between buildings, nor between finish treatments (BS, BF, NS, or NF). No differences were apparent in MC of siding on different wall constructions, nor was there an apparent effect of wall orientation. Furthermore, siding boards on the bottom course were not wetter than siding boards in other locations. However, most of the moisture pin data were close to the lower measurement limit of the instrumentation, hampering our ability to discern actual differences in moisture content.

3.2.2 Final moisture contents

Gravimetric MC measurements indicated that final in-place siding MC was fairly low. Siding moisture content on the two buildings was essentially the same: Specimens from the guttered building had MC's in the range of 5% to 8%, with an average MC of 6.4%. Specimens from the building with overhangs also had MC's in this range, with an average of 6.3%. Final moisture contents of siding in bottom course positions were the same as the average moisture content of boards in other courses². This is in agreement with moisture pin data.

Sorting the data by wall orientation and finish treatment generally reduced variability in the data, suggesting systematic influence of these variables on final in-place MC. We used group t-tests on the sorted data to identify if orientation (by building and finish treatment), or finish treatment (by building and wall orientation), had a statistically significant influence on final in-service MC. In most cases wall orientation had a statistically significant effect on final in-place siding MC (confidence coefficient of 0.95), with boards on the sides of the building that received more direct sunshine generally drier. This was in agreement with thickness measurements made at quarterly inspections. With the data sorted by wall orientation, t-test comparisons also revealed that, at the time of removal, boards not backprimed were drier than backprimed boards (confidence coefficient of 0.95) in roughly half of the comparisons, while there were no cases where backprimed boards were drier than boards not backprimed.

Average MC of siding specimens taken from below windows that showed evidence of leakage was 6.8%. One specimen which had water stains on its back surface, registered 9% MC. The rest of the specimens were 6 or 7% MC (including another specimen with water stains on its back).

We found that wall construction (i.e. presence or absence of plywood sheathing or type of secondary weather barrier) had no significant effect on final MC of the siding.

3.4 Rainwater leakage at windows

We found water stains on the back of siding below 6 of the 12 windows on the building with gutters (without overhangs). Four of these six windows were on gable-end walls. The leakage occurred between the window units and the trim, or between individual trim pieces. Although the water stains on siding backs indicated that some water had leaked into the siding system, there was no evidence of damage to the

² The buildings were surrounded by a roughly 18-inch wide margin bed of gravel and sand, which probably had substantial drainage potential.

siding. Water stains were limited to the three courses of siding directly below the window. Tracing of stains over the backs of successive siding courses indicated that in some cases the water drained to the front of the siding at the laps; in other cases the water apparently evaporated. On the building with overhangs we did not see staining of siding backs below windows.

Areas below windows were the only places with evidence of rainwater leakage into the siding system. The secondary weather barrier (#15 felt or polyolefin) did not show evidence (wrinkling, or staining) of exposure to water.

3.5 Moisture contents of wood trim

Most of the moisture meter measurements of wood trim, including pine door jambs and cedar trim, taken in May 1997, indicated moisture contents between 9 and 12 percent. One reading was over fiber saturation; this reading was in the trim of a window where subsequent siding removal indicated leakage had occurred. A few readings in trim on gable end walls were over 20 percent; they were taken at the lower end of door jambs (three readings), near the ends of window trim (three readings) or near the end of gable end trim (one reading). One lower end of a door jamb was decayed. Table 3 shows the distribution of all MC readings by building.

Table 3. Distribution of MC readings on wood trim taken in May 1997

MC readings Range	Building with Gutters Number of readings	Building with Overhangs Number of readings
MC $\geq$ 30%	1 (0.6%)	0
25%#MC<30%	1 (0.6%)	0
20%#MC<25%	3 (1.7%)	3 (1.6%)
15%#MC<20%	20 (11.6%)	12 (6.6%)
10%#MC<15%	122 (70.9%)	150 (82.4%)
MC<10%	25 (14.5%)	17 (9.3%)
Total	172 (100%)	182 (100%)

Compared to the building with gutters, the wood trim on the building with overhangs had fewer high moisture content readings, with none over 22% MC. It appears that the overhang provided protection from rain and reduced the moisture content variations in the wood trim.

We sorted the moisture meter data for window and door trim, including pine door jambs, by gable-end wall versus eave wall. The data is shown in Table 4. With one exception, moisture meter readings on window and door trim were taken in identical locations on each building. We performed paired t-tests on the data (paired by individual location) to compare trim MC's between buildings. On gable-end walls, the difference in trim MC's between buildings was highly significant (confidence coefficient of 0.995). Thus roof overhangs appeared to help keep window and door trim dry on gable-end walls. On eave walls however, the difference between buildings was not significant. This suggests that rain gutters are as

beneficial as roof overhangs, provided the gutters are kept clear of obstructions.

Table 4 Average MC of window and door trim by building and wall type

Type of wall	Average MC (%)	
	[Std. dev. (% MC)/number of specimens]	
	Building with gutters	Building with overhangs
Gable-end	12.8 [4.1/78]	11.6 [2.3/78]
Eave	12.0 [1.9/54]	11.7 [1.7/54]

4 Discussion

This particular hardboard siding remained dry and showed essentially no non-recoverable thickness swelling in service in a very warm and humid climate. In-service siding MC ranged from approximately 6% to 8% MC. This is roughly equivalent to the sorption EMC of this material for 55% to 75% RH. The lack of backside staining (except at windows) or of evidence of wetting of the secondary weather barrier suggests that wetting by wind driven rain at siding laps either did not occur or did not occur to any appreciable degree.

A limited amount of water leakage occurred in at least six of the 24 windows between the window units and the trim, and between individual pieces of trim. Prevention of water intrusion at these locations relied on the long-term performance of bedded and fillet caulk seals, which are not the most reliable type of sealant joints. Evidence of water leakage of this type was much more prevalent in the building without overhangs. The leakage occurred even though window installation, flashing, and caulking were done carefully. In some cases the lap siding permitted drainage of at least part of this leakage to the exterior. The leakage did not affect siding performance, but in some cases led to elevated MC of the cedar window trim and to fastener corrosion.

The relative prevalence of window leakage, and the final in-place MC of trim, suggest that roof overhangs reduced moisture stress on exterior millwork and trim but there is no indication that overhangs influenced the MC of the siding.

Finishing treatments used in this study had no visible influence on the condition of this siding. Final in-place MC of backprimed siding was not lower than that of siding without backpriming, but was sometimes higher. The full paint coverage over the entire backs of backprimed specimens apparently retarded drying of the siding through the back surfaces.

To obtain a better understanding of the effect of backpriming under cyclical conditions, we placed four 203 mm (8 in) square specimens (retrieved from the test buildings) in humidity rooms and monitored their MC. The rooms were at 27°C (80°F) and at 30% and 90% RH, respectively. Samples remained in one room for two weeks and were then moved to the other room. Figure 3 shows drying and wetting curves for the first full exposure cycle, along with MC's at the end of each two-week exposure period during the second cycle.

Figure 3 shows that backpriming significantly retarded both vapor sorption and desorption, i.e. backpriming retards both wetting and drying. Site-collected data suggest that heavy rains occurred in the months prior to siding removal, and dry weather the week prior to removal. At the time of removal backprimed siding consistently had a slightly higher MC possibly because it had not yet fully dried.

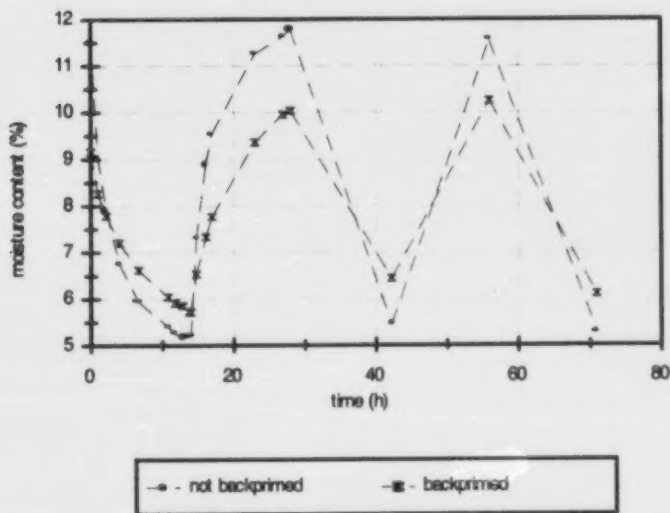


Fig. 3: Average MC of siding with, and without backpriming, when exposed cyclically to 30% and 90% RH

The residual edge swell and linear expansion properties of the siding used in this study was much better than minimum requirements in the product standard (ANSI/AHA, 1990). Biblis (1989, 1991) indicates that quality of commercial hardboard siding, as judged by performance in laboratory test procedures, can vary substantially, with some markedly inferior to that used in this study. Our observations therefore cannot be extended to all commercial hardboard siding.

5 Conclusions

Our observations apply to siding from one particular mill with properties substantially exceeding the minimum industry requirements, and installed according to manufacturer's recommendations.

- 1 All the siding used in this study remained in excellent condition after 29 months of exposure on test buildings in southern Florida, even though some water leakage at windows was not uncommon. Weather conditions during exposure

were close to historic norms for the location; annual rainfall was close to 60 inches per year, an extreme exposure for the continental United States.

- 2 The siding remained at 9% MC or less over the exposure period.
- 3 Water leakage at windows, apparently caused by less-than-perfect window-to-trim sealing details, was not uncommon, even though installation, flashing, and caulking were done carefully. This supports the contention that window leakage is common and that the cladding must therefore accommodate some degree of water leakage.
- 4 Roof overhangs had no apparent influence on MC of this siding, but did lower MC in exterior millwork and trim. Gutters, when kept unobstructed, had a similar effect.
- 5 Backpriming of the siding did not improve performance of this siding, nor did it result in lower in-service MC. Final in-place MC of backprimed siding was sometimes a little higher than that of siding not backprimed, but this could have been caused by slower drying of the backprimed boards after a period of rain. Because siding that was not backprimed had some paint coverage on the backs at the overlaps, our conclusion that backpriming was not beneficial may not apply when the comparison is made with siding that is completely bare on the back. The conclusion may also not be applicable to situations where backpriming is limited to a "border" strip at the siding overlap.

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65 PREDICTION OF THE IMPACT OF THE ENVIRONMENT ON TIMBER COMPONENTS: A GIS-BASED APPROACH

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Abstract

This paper reports on a system for the prediction of moisture content of timbers and corrosion of connectors in the building envelopes of Australian houses. The system forms part of a larger method being developed to facilitate reliability-based design of timber structures. The system is constructed using geographic information system (GIS) linked to climatic databases. Rules are developed which permit the sequential estimation of moisture content of air in sheltered external environments, the moisture content of specific timbers in this same environment, the moisture content of timbers in different spaces in the building envelope and, lastly, the corrosion of connectors in such timbers. The accuracy of the method and its limitations are discussed.

Keywords: Houses, timber, moisture content, building envelope, connectors, corrosion

1 Introduction

The work reported in this paper forms part of a larger project with the aim of developing a reliability-based method for designing durable timber structures (Foliente et al. 1999). If a design method is to be derived for timber structures, it is necessary to estimate the moisture content of timber members in a structure. The moisture content will have a profound effect on the probability of decay of timber members and corrosion of timber connectors. In this paper, a method for the prediction of moisture content of timber components in houses is presented. To illustrate the utility of such data, the moisture content estimations are then combined with models of connector corrosion to develop a system for estimating connector life.

2 Overview of a GIS-based model

A computer model for the prediction of moisture content in timber and the corrosion rate of timber connectors has been derived. This model combines a system of rules and climatic databases with a geographic information system (GIS) representation. The model aims to estimate the moisture content of and lifetime of connectors for timber members within building spaces of standard houses in any location in Australia. To do this, the model performs the following functions:

- Meteorological data is 'generalised' and the generalised data is used to estimate the nominal surface moisture content (NSMC) (%) of an 'ideal' timber in a sheltered external location at any geographical location in the country.
- Using experimental rules derived from chamber tests, the moisture content of specific timbers in sheltered external locations (ShMC) is then estimated from the NSMC.
- Another series of rules derived from the house monitoring program (Cole 1996) is then used to estimate the moisture content of timbers in specific building envelope locations (BMC) from the ShMC.
- Experimental relationships between the corrosion rate of connectors, moisture content and timber properties are then used to estimate the corrosion rate in given building envelope cavities from data on timber properties and BMC.

3 Prediction of nominal surface EMC

The nominal air moisture content is calculated from RH and temperature as measured at meteorological stations using the approximation of Bramhall (1979). It is assumed that under sheltered (from both sun and rain) external exposures, the surface of a timber component will be in equilibrium with the air and thus Bramhall's equations can define NSMC. A three-hourly climate database from the Bureau of Meteorology is used to calculate the NSMC of timber in a sheltered external exposure. The value of the climatic variable at geographic positions between meteorological stations was estimated using a triangular irregular network, or TIN, model following the procedures of Okabe et al. (1992).

4 Prediction of moisture content of selected timbers

The estimated moisture content for a particular timber is calculated as:

$$\text{ShMC}_i = A_i + B_i \times \text{NSMC} \quad (1)$$

where A_i and B_i are derived regression constants for each particular timber species, and ShMC_i is the moisture content of the particular species (in a sheltered external location). The regression constants were derived from results of actual experiments. The values of A and B for selected timbers are given in Table 1, where values for similar timbers are grouped together.

Table 1: Constants for NSMC/ShMC relationships

Timber	A	B
Huon pine	-1.18	0.81
Oregon	-1.97	0.87
Kauri	-3.79	1.06
Spotted gum	1.83	0.63
Red iron bark/mountain ash	-3.23	1.00
Brush box	-3.33	0.96
LOSP/RP	-2.46	1.05
H3/H5	-2.67	1.12
Douglas fir/karri	-2.67	0.99
Meranti	-2.91	0.92

5 Prediction of moisture content in a given position

The estimated moisture content value can be further adjusted using micro-environmentally based adjustment factors. The average seasonal moisture content in a given building space (BMC_{sa}) is given by an adjusted value and is computed as follows:

$$BMC_{sa} = ShMC_{aa} + D*(ShMC_{sa} - ShMC_{aa}) + \Delta_{sea} + \Delta_{climate} + \Delta_{radiation} \quad (2)$$

where the subscript sa and aa refer to the seasonal average and annual averages respectively. Equation (2) indicates that the seasonal variation in the moisture content of a piece of timber in a building envelope space is damped with respect to the sheltered exterior. The damping factor D is still under investigation, but for facades is approximately 1 (i.e. no damping) and for subfloors, wall cavities and roof spaces is 0.5.

The coastal factor $\Delta_{sea} = +2$ applies to all external locations within 1 km of the coastline and reflects the higher RH of adjacent air and the effect of the hygroscopicity of salt deposited on timber surfaces. The solar radiation factor $\Delta_{radiation}$ applies to all external locations, where $\Delta_{radiation} = -2.5$ and -1 for areas with high and low solar radiation respectively. The magnitude of solar radiation is based on interpolated solar radiation level using a Delaunay tessellation model (Okabe et al. 1992). The climatic factor $\Delta_{climate}$ applies to all locations. The value of the climatic factor is given in Table 2. These parameters are based on observed variations between timbers exposed in the given locations.

6 Corrosion of nails

The corrosion rate of nails embedded in a particular timber is estimated depending on the type of timber and nails. Two types of nails were considered in this study, namely, bright nails and hot-dipped (zinc coated $\geq 600 \text{ g/m}^2$) nails. The estimation procedure assumed that the nail corrosion rate will increase in a linear mode with increasing moisture content until, at a critical moisture content, the corrosion rate remains constant with further increases in moisture content.

The proposed relationship between corrosion rate and equilibrium moisture content (EMC) of timber is as follows:

- The maximum corrosion rate is reached when a certain EMC in the timber is reached. This maximum threshold value depends on the particular timber.
- No corrosion is assumed to occur if the EMC level is below some minimum threshold value which is also dependent on the particular timber.
- At EMC values between the minimum and maximum threshold values, the corrosion rate is assumed to vary with EMC.

Table 2: Values of Δ_{climate} for different climatic zones and building spaces

Zone	Facade	Subfloor	Wall cavity	Roof space
Tropical	1	1.2	0	-3
Subtropical	1	1	-1	-3
Temperate	1	1	-1	-3
Inland	1	1	-1	-3
Alpine	2	1	0	-3

The form of the corrosion/EMC dependence was determined by experimentation for the timbers listed in Table 1. The form of the dependence has been derived by a regression analysis using a series of experiments. Timber samples were conditioned to set RH and temperature conditions until the moisture contents of the timbers equilibrated (at three different moisture contents from 14 to 24%), at which stage two timber panels were nailed together and conditions were maintained for 120 days. Hot-dipped coated and bright (uncoated) steel nails were used. It was found that for bright nails, the timbers could be grouped into three classes, as given in Table 3. These classes are defined purely on the basis of observed performance, such that:

- Class 1 – when embedded in these timbers, bright nails show a plateau in corrosion rate above 16–18% moisture content.
- Class 2 – when embedded in these timbers, bright nails show a plateau in corrosion rate above 20–24% moisture content.
- Class 3 – when embedded in these timbers, bright nails do not show a plateau in corrosion rate with increasing moisture content.

For corrosion of hot-dipped nails, the same classification could be used. For each timber used, the pH, conductivity and acid content of water in equilibrium with the timber was determined using a method adapted from Krilov and Lasander 1988. This data was then used to develop the regression models. The corrosion rate ($\text{g/m}^2\cdot\text{year}$) of the plateau (Corr_{max}) was determined to be:

- For class 1 timber and bright nails

$$\text{Corr}_{\text{max}} = 4.5 + 2.97 \times \exp \left[2.380 + \left(\frac{D_d}{500} \right)^4 \right]$$

For class 2 timber and bright nails

$$\text{Corr}_{\max} = 0.8 + 3.0 \times \exp \left[\begin{aligned} & -0.526 - 0.238 \times \left(\frac{\text{CH}_3\text{COOH}}{300} \right)^{1.3} \\ & + 0.531 \times \ln(\text{Conductivity}) + 0.251 \times \left(\frac{D_d}{500} \right)^{3.2} \end{aligned} \right]$$

- For class 1 and 2 timbers for hot dip nails

$$\text{Corr}_{\max} = 0.57 + 3.0 \times \exp \left[\begin{aligned} & -3.053 + 0.45 \times \left(\frac{D_w}{100} \right)^{3.2} \\ & -0.254 \times \left(\frac{D_d}{500} \right)^{2.9} + 1.17241 \times \ln(\text{Conductivity}) \end{aligned} \right]$$

In these equations, D_d is the dry density of the timber, D_w is the density of water in the timber, pH, Conductivity and CH_3COOH are the pH, conductivity (in μS) and the acid concentration (in meq/100 g of wood), assuming the acid is CH_3COOH of water in equilibrium with the timber.

Table 3: Form of moisture content/corrosion dependence

Class	Condition	Timbers
1	Plateau in corrosion rate at 16–18% MC	Brush box, meranti, spotted gum, huon pine, karri, red iron bark, LOSP-treated pine
2	Plateau in corrosion rate at 20–24% MC	Douglas fir, mountain ash, radiata pine
3	No plateau	CCA-treated timbers

At moisture contents below which the maximum corrosion rate occurs, the corrosion rate as a percentage of the maximum is given as:

- For bright nails in class 1 timbers

$$\text{Corr}\% = \begin{cases} 28.8051 + 0.0546587 \times \text{EMC}^{2.5} & 12.4 \leq \text{EMC} \leq 22.3 \\ 0 & \text{EMC} < 12.4 \end{cases}$$

- For bright nails in class 2 timbers

$$\text{Corr}\% = \begin{cases} 100 \times [0.02 + 0.071 \times (\text{EMC} - 4)] & 10 \leq \text{EMC} \leq 16.5 \\ 0 & \text{EMC} < 10 \end{cases}$$

- For hot-dipped nails in class 1 and 2 timbers

$$\text{Corr}\% = \begin{cases} 100 \times [0.55 + 0.006 \times (\text{EMC} - 14)] & 14 \leq \text{EMC} \leq 22 \\ 0 & \text{EMC} < 14 \end{cases}$$

For CCA-treated timbers, a plateau is not reached and so only one equation is given, viz:

- For bright nails

$$\text{Corr} = 0.63 + 2.98 \times \exp [49.2394 - 5.7525 \ln(D_d) - 20.332 \ln(\text{pH}) + 3.994 \ln(D_w)]$$

- For hot-dipped nails

$$\text{Corr} = 0.45 + 3.0 \times \exp \left[3.315 + 0.126 \times \left(\frac{D_w}{100} \right)^{3.0} + \left(\frac{0.152 \times \text{Conductivity}}{200} \right)^{2.9} \right]$$

The above equations define the initial corrosion rate, however it is well established that for atmospheric corrosion, the corrosion rate decreases with time. It is postulated that this is also the case for metals in timber and that the corrosion (Corr) after a given time t (in years) is:

$$\text{Corr} = A \times t^n$$

where A is derived from the corrosion rates defined above (given that these rates were determined over a 4 month exposure). The exponent ' n ' is a constant with initial analysis indicating that an appropriate value for n is 0.5, except for exposures to marine environments when $n = 0.7$ is more appropriate. When the corrosion in time t is determined, the life is defined by when the permissible average thickness loss is equal to the average corrosion.

7 Comparison of experimental and predicted moisture content

Field data from moisture sticks exposed along the eastern seaboard in sheltered external locations and in ventilated regions of subfloors provides a basis for validating the predictions of the model (see Cole (1996) for detailed procedures). Timbers exposed in this program were brush box, spotted gum, Douglas fir, treated pine - H3, radiata pine and mountain ash.

7.1 Prediction of surface moisture content

The GIS system predicts the surface moisture content from meteorological (met) stations data. In most cases, a given dwelling will be at a considerable distance (up to 50 km) from the nearest met station, and thus there may be significant differences between the local climate and therefore the predicted surface moisture content at the dwelling and the met station. In Table 4, the predicted NSMC at a number of sites and at the nearest met station are given. The sites used to calibrate the model are Innisfail 1, Sydney Narrabeen, Mt Buller and Narrandera. Innisfail (lat. 17.525, long. 145.986) is located on the coast in tropical north-east Australia, Narrabeen (lat. 33.708, long. 151.302) is adjacent to a surf beach in the Sydney metropolitan area and thus has a temperate climate, Mt Buller (lat. 37.142, long. 141.450) is a ski lodge in the alpine region of southern Australia, and Narrandera (lat. 34.748, long. 146.547) is inland in southern Australia and has a savanna-type climate.

Table 4: Surface moisture content (%) at sites and met stations

Site	Autumn	Winter	Spring	Summer	Average
Innisfail	19.3	18.6	15.8	17.9	17.8
Innisfail met	18.2	17.5	15.7	17.1	17.0
Brisbane Gap	16	18	16	15	16.5
Brisbane met	15	16	15	14	15.2
Sydney Narrabeen	18.6	17.4	16.8	17.3	17.6
Sydney met	15.2	14.2	14.8	15.7	14.9
Mt Buller site	21.2	22.4*	21.2	17.3	20.5
Mt Buller met	16.7	20.4	19.1	15.2	18.4
Narrandera site	13	14	19	16	17.3
Narrandera met	12	17	23	17	19.6

The RH and temperature used to predict NSMC at the sites is measured in exterior sheltered locations on the facades of buildings (under eaves etc.), while at met stations these same parameters are generally measured in the open (though enclosed in a Stevenson screen). The surface moisture content at the sites is in general 1 to 2 percentage points below the values at the met stations. In some cases, this may be associated with differences in local climate, for example in Brisbane 'The Gap' is in the Brisbane Hills while the met station is on flat land near the Brisbane River. Thus, higher RH would be expected at The Gap. Innisfail is at O'Briens Hill, on higher ground (outside town) than the met station and thus could have a different microclimate.

However, a microclimate will also exist on the facade of a dwelling (compared to an open exposure) where wind and sunlight will be reduced. Thus, a facade correction factor (of +1% for all climatic zones except Alpine which is +2%) was introduced. The observation that marine sites such as Narrabeen, which is on a northern Sydney beach, have NSMCs of more than 3 percentage points higher than met stations away from the coast (e.g. Sydney met) justifies the additional marine correction factor. This factor may be associated with humidity carried by sea breezes.

7.2 Actual and predicted moisture content

7.2.1 Moisture content of boards exposed in sheltered exteriors

In this section, the measured and predicted moisture contents are compared. Moisture content was scheduled to be measured every three months over a two-year period. The predicted moisture contents are based on ten-year climatic data, and thus some variations between measured and predicted data could be associated with variations between the particular two years (1996-1998) during which the measurements were made and the long-term average data. In Table 5 the experimentally measured moisture content (MMC) for brush box is given, while in Table 6 the predicted moisture content (PMC) is given for the same timber. The Sydney marine prediction should be compared with Sydney Narrabeen.

Comparisons such as given by Tables 5 and 6 have been made for all timbers, however to conserve space this data has been summarised in Table 7. This table presents a comparison between the annual average measured and predicted moisture content, i.e.:

$$\text{Accuracy} = \text{MMC}_{aa} - \text{PMC}_{aa} \quad (3)$$

where aa signifies the annual average.

7.2.2 Moisture content of boards exposed in ventilated subfloors

In this section, measurements made in ventilated subfloors of selected houses are presented and compared with predictions. Innisfail 1 is a brick veneer house with a suspended floor having a clearance of 0.3–1.8 m. The Gap is brick veneer with a very variable clearance of 0.2–1.5 m. Mt Buller is a concrete block, suspended floor construction with clearance of 0.3–0.5 m.

In Table 8, the MMC for brush box is given for the same timber. In Table 9, the accuracy of the predictions (using Eq. (3)) are given for all timber types.

Table 5: MMC (%) of brush box on sheltered facades

Site	Autumn	Winter	Spring	Summer	Mean
Innisfail 1	14.2	14.6	13.5	14.3	14.1
Brisbane Gap	13.8	14.6	13.4	13.7	13.9
Sydney Narrabeen	16.5	16.0	13.3	17.4	15.8
Mt Buller	14.2	14.4	15.9	13.0	14.4

Table 6: PMC (%) of brush box on sheltered facades

Site	Autumn	Winter	Spring	Summer	Mean
Innisfail	14.1	15.2	14.5	12.9	14.1
Brisbane	13.1	12.3	11.6	12.3	12.3
Sydney marine	15.4	16.8	16.0	15.4	16.1
Mt Buller	14.8	18.3	13.6	11.3	14.5

Table 7: Summary of accuracy of predictions of MC on sheltered facades

Location	Brush box	Spotted gum	Douglas fir	Radiata pine	Treated pine	Mountain ash
The Gap	1.6	-0.8	0.6	0.1	0.6	1.5
Innisfail 1	0	-0.3	-0.7	-0.3	-0.3	0
Narrabeen	-0.3	-2.9	0	0.8	0.1	-0.5
Mt Buller	-0.1	-0.2	-0.1	1.3	-0.9	0.3
All	0.5	-0.8	0.2	0.3	-0.5	0.6

Table 8: MMC (%) for brush box in ventilated subfloors

Location	Autumn	Winter	Spring	Summer	Mean
Innisfail, measured	15.1	15.3	14.6	15.1	15.0
Innisfail, predicted	14.3	14.8	14.5	13.7	14.3
Gap, measured	13.2	13.4	13.0	13.4	13.3
Gap, predicted	12.7	12.3	12.0	12.3	12.3
Mt Buller, measured	14.4	14.5	14.3	13.9	14.3
Mt Buller, predicted	14.6	16.4	14.1	12.9	14.5

Table 9: Summary of accuracy of predictions of MC in ventilated subfloors

Location	Brush box	Spotted gum	Douglas fir	Radiata pine	Treated pine	Mountain ash
Innisfail 1	0.7	-0.2	-0.4	-1.5	-0.9	0.6
Innisfail 2	1.6	0.5	0.4	-0.5	-0.4	0
The Gap	-0.1	-0.9	0.6	-1.7	0.1	-1.0
Mt Buller	-0.2	0.6	1.2	0.1	0.5	1.4
All	0.2	-0.4	0.3	0.3	-0.5	0.6

8 Accuracy of the prediction of the corrosion rate of nails

Table 10 shows the measured and predicted corrosion rates for hot-dipped nails on the facade for a range of timbers at Mt Buller, Narrabeen and Brisbane Gap.

From Table 10 it is again evident that the models are reasonably accurate at predicting the moisture content of the timber. Further, the model was in general reasonably accurate at predicting the corrosion rate of hot-dipped nails. There are however a number of exceptions (e.g. the predicted corrosion rate of hot-dipped nails in radiata pine at Brisbane Gap is much higher than measured) which are still being investigated.

Table 10: Comparison of predicted and measured moisture contents and corrosion rates on sheltered facades

Location	Timber	Measured moisture content (%)	Predicted moisture content (%)	Measured corrosion rate (g/m ² .year)	Predicted corrosion rate (g/m ² .year)
Mt Buller	H5	15	15	19	35
	H3	15	15	9	8
	LOSP	15	15	9	8
	Brush box	11	13	27	10
	Douglas fir	14	15	21	34
Brisbane Gap	Spotted gum	13	12	11	9
	Brush box	11	13	5	7
	Radiata pine	15	15	7	34
Sydney	H5	15	15	70	71
Narrabeen	H3	16	16	38	56
	LOSP	14	15	34	13
	Douglas fir	14	14	57	53

9 GIS representation of data

The GIS system can be used to develop maps to define critical parameters. In Figure 1, a map is given of the per cent of time that the NSMC on an ideal timber on the facade (sun and rain sheltered) on a building exceeds 16%. This map gives an indication of regions in which corrosion or decay may occur.

10 Discussion

10.1 Accuracy of moisture content prediction

The models presently formulated do not include terms to take into account the rate of diffusion of moisture into timber. An analysis of the accuracy of the predictions indicates that the prediction of monthly or seasonal average moisture contents do not require this level of analysis. Future versions of the model will include such terms in order to predict daily or weekly variations in moisture content. The level of detail regarding moisture content variations (in time and in section) required to reasonably model the degradation process have yet to be determined.

The method used to predict the moisture content of air in building envelope cavities compared to external conditions is at present largely empirical. It appears to reasonably model the Australian data. However, the data set is quite small and is related to specific materials and designs. Work is being carried out taking into consideration heat and mass transfer to derive more fundamentally based relationships between external and building envelope conditions.



Fig. 1: Variation of time above 16 EMC in sheltered facades across Australia

However, despite the limitations of the approach outlined above, the method appears reasonably accurate. The major source of errors appears to be the prediction of conditions at the house site compared to the nearest meteorological stations. Approximation procedures that combine meteorological and local terrain data to estimate local climate are currently under investigation.

11 Conclusion

The GIS-based system has the capacity to predict with reasonable accuracy:

- Moisture content of a timber member in a standard Australian house.
- Probable life of connectors in timbers in a standard Australian house.

The development of the system has highlighted areas for further development.

12. References

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66 MATHEMATICAL MODELLING OF SORPTION PROCESSES IN TIMBER, BASED ON NUMERICAL ANALYSIS

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Abstract

Although moisture is a leading cause of degradation in a range of building materials the transport mechanisms for adsorption and desorption are not well understood and mathematical modelling has been of limited success. This paper presents a mathematical model with numerical analysis by use of the method of finite differences. Mathematical analysis of moisture transmission with assumption of Newmann's boundary conditions was made. The model can be used to describe the process of adsorption and desorption of moisture in timber, where the transport of moisture follows the transverse direction of wood, with potential extension to the longitudinal direction also. The model makes it possible to attain the profiles of concentration of moisture developed throughout the sample. Data resulting from changes in relative humidity and collected by continuous monitoring by calibrated moisture sensors are compared against result from modeling. Good agreement has been observed over various conditions.

Keywords: sorption model; timber moisture content, numerical analysis

1 Introduction

Moisture is associated with a range of problems in connection with durability and degradation in a range of building materials. In particular, for wood, warping, shrinkage and irreversible distortion in service can occur where substantial variations exist between the installation and final moisture content (Ahmet 1995 and 1996). The resistance to decay is severely affected by the moisture content (MC) of timber as the colonisation of fungi is strongly dependent on the MC. For example, if the moisture content is above around typically 22% the probability of fungal decay increases dramatically, especially where the timber is classed as "non-durable" (TRADA 1991). Various studies, such as Tong and Suchsland 1993, show that the transient warp may not be recovered after

eliminating the moisture gradient, and may contribute to permanent deformation of the panel. It is also found that differences in moisture content through the thickness of the timber are responsible for the differential shrinkage which leads to timber degradation. Therefore, analysis of the warping and shrinkage problems requires knowledge not only of the equilibrium moisture content (EMC) corresponding to a given relative humidity (RH) and temperature, but also of the time-dependent moisture distribution within the wood.

Despite the range and frequency of the problems encountered the current understanding of moisture transport mechanisms in timber is limited. Nevertheless numerous mathematical models have been published to describe the wood moisture movement (Plumb 1985; Stanish 1986; Liu 1994; Pang 1996). However, due to the complexity of the moisture transfer phenomenon there has been much debate as to the methodology and form of an appropriate sorption model. The resulting publications, although interesting to the academic, often fall short of their potential due to, for example, a lack of experimental data, oversimplification, or unknown coefficients. In most cases, model builders use their own experimental data to fit their models. Problems arise where the sorption data are erroneous or inaccurate as the reliability is greatly affected by the accuracy of measurement of equilibrium moisture content and temperature.

This paper presents a mathematical model and a well-tested experimental system to describe and collect reliable data respectively from both the processes of adsorption and desorption.

Since the CIB World Building Congress presentation (Dai and Ahmet 1998) the moisture sensors which were at that time under development have been improved to the extent that they have become a very reliable tool for moisture monitoring. For testing our mathematical model, continuous measurement of moisture content as a function of both depth and time have been achieved by use of these sensors. The data also showed that resistivity monitoring method which links with moisture content is highly suitable for monitoring moisture content field in moisture transmission.

2 Theory

2.1 Equations and analysis

The transfer of moisture within the wood can be described by the second Fick's law:

$$\frac{\partial C(x,t)}{\partial t} = D \frac{\partial^2 C(x,t)}{\partial x^2} \quad (0 < x < a, t > 0) \quad (1)$$

where C is the moisture content at the position x and time t , a is the sample thickness and D is the diffusion coefficient (described further in Section 2.3).

The assumptions for its use are as follows:

1. Constant diffusivity i.e. diffusion coefficient D is constant.
2. Moisture is transferred through the transverse direction.
3. Moisture gradient as a driving force of moisture movement.
4. Newmann's boundary condition is applied in this study; i.e. the rate at which moisture is emitted through the wall surface at any time is equal to the rate at which moisture diffuses from the interior to the surface or *vice versa*.

Let the initial condition be

$$C=C_1 \quad (0 < x < a, t > 0) \quad (2)$$

where C_1 is a constant concentration in the medium. The boundary conditions could then be mathematically written as:

$$\frac{\partial C(0,t)}{\partial x} = 0 \quad (3)$$

$$D \frac{\partial C(a,t)}{\partial x} = K[C_2 - C(a,t)] \quad (4)$$

where K is the coefficient of moisture transfer at surface and C_2 is equilibrium concentration corresponding to the vapour pressure in the atmosphere remote to the surface.

To obtain more general results, dimensionless parameters are defined. Then, equations (1) to (4) could be further written in two different dimensionless formulation in two cases for sorption when $C_2 > C$ and for desorption when $C_2 < C$.

2.1.1 Adsorption

Let $E(\xi, \omega) = [C(\xi, \omega) - C_1] / [C_2 - C_1]$, $\xi = x/a$, $\omega = Dt/a^2$, $\psi = Ka/D$. Here the dimensionless parameter E , ξ and ω represent moisture content, thickness and time in the form of Fourier number, respectively. ψ is dimensionless ratio of surface and internal resistance on surface. Thus, equations (1) to (4) can be put in dimensionless form in the same order as previously presented:

$$\frac{\partial E(\xi, \omega)}{\partial \omega} = \frac{\partial^2 E(\xi, \omega)}{\partial \xi^2} \quad (0 < \xi < 1, \omega > 0) \quad (5)$$

$$E = 0 \quad (0 < \xi < 1, \omega = 0) \quad (6)$$

$$\frac{\partial E(\xi, \omega)}{\partial \xi} = 0 \quad (\xi = 0, \omega \geq 0) \quad (7)$$

$$\frac{\partial E(\xi, \omega)}{\partial \xi} = \psi[1 - E(\xi, \omega)] \quad (\xi = 1 - \epsilon, \epsilon \rightarrow 0, \omega > 0) \quad (8)$$

2.1.2 Desorption

Let $E(\xi, \omega) = [C(\xi, \omega) - C_2] / [C_1 - C_2]$, $\xi = x/a$, $\omega = Dt/a^2$, $\psi = Ka/D$. Thus, equations (1) to (4) can be put in dimensionless form in the same order as previously presented:

$$\frac{\partial E(\xi, \omega)}{\partial \omega} = \frac{\partial^2 E(\xi, \omega)}{\partial \xi^2} \quad (0 < \xi < 1, \omega > 0) \quad (9)$$

$$E = 1 \quad (0 < \xi < 1, \omega = 0) \quad (10)$$

$$\frac{\partial E(\xi, \omega)}{\partial \xi} = 0 \quad (\xi = 0, \omega \geq 0) \quad (11)$$

$$\frac{\partial E(\xi, \omega)}{\partial \xi} = -\psi E(\xi, \omega) \quad (\xi = 1 - \epsilon, \epsilon \rightarrow 0, \omega > 0) \quad (12)$$

2.2 Electrical properties

Skaar (1988) reported regression equations of resistivity R against moisture content M of the form

$$\log R = A - B \log M \quad (13)$$

for a group of woods for measurement made along the grain over the moisture content range from 8% to 20% at 20°C. The typical mean values of A and B of equation (13) are available in Skaar (1988). In this study, the coefficients were determined based on the oven-dried data set up in TRADA (1998).

2.3 Diffusion coefficient determination

The diffusivity D (mm²/hr) was calculated from the slope of the linear portion of the sorption curve (Siau 1984):

$$D = \left(\frac{\pi a^2}{16} \right) \left[\frac{M_2 - M_1}{\sqrt{t_2} - \sqrt{t_1}} \right]^2 \quad (14)$$

where M is the percentage moisture content at time t (measured in hour) based on the resistance in equation (13); subscripts 1 and 2 represent two points on the linear part of the sorption curve. In calculating the diffusivity, the values of t_1 and t_2 were taken to correspond to $M=0$ and $M=0.5$, respectively (Crank 1975). Moisture content at the two-third saturation point for RH was calculated and assigned to the diffusion coefficient. The data were fit to an exponential function (Simpson 1993) through regression analysis:

$$D = D_r \exp(GM_{2/3}) \quad (15)$$

where D_r is reference regression constant; G is regression constant and $M_{2/3}$ is moisture content at the two-third saturation point of a given RH interval.

2.4 Numerical technique employed

The equations (5) to (8) or (9) to (12) are solved numerically by an explicit forward difference method. The wood sample is divided into a number of cells of constant thick $\Delta \xi$ in which the moisture content $E_{i,j}$ is known at the time $i\Delta\omega$, see Fig. 1. Considering variable mesh spacing, the appropriate second order finite

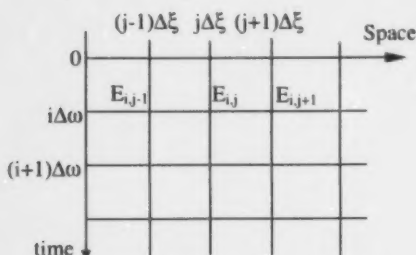


Fig. 1: The dimensionless space-time numerical mesh

difference approximations of the spatial derivatives of a function E at a point (i) are:

$$\left(\frac{\partial E}{\partial \omega}\right)_i = \left(\frac{\Delta E}{\Delta \omega}\right)_i = \frac{E_{i+1} - E_{i-1}}{\Delta \omega_+ + \Delta \omega_-} \quad (16)$$

$$\left(\frac{\partial^2 E}{\partial \xi^2}\right)_i = \left(\frac{\Delta^2 E}{\Delta \xi^2}\right)_i = \frac{2}{(\Delta \xi_+)(\Delta \xi_-)} \left(\frac{\Delta \xi_-}{\Delta \xi_+ + \Delta \xi_-} E_{i+1} - E_i + \frac{\Delta \xi_+}{\Delta \xi_+ + \Delta \xi_-} E_{i-1} \right) \quad (17)$$

where $\Delta \xi_+$ indicate the mesh size to the right of the grid point i , and so on and so forth.

2.5 Comparison with analytical solution

Considering the sorption case, under conditions of constant surface temperature and with constant diffusivity, the solution of the equations (5) to (8) for above boundary conditions can be found in Crank (1975) as:

$$E(\xi, \omega) = 1 - 2\psi^2 \sum_{n=1}^{\infty} \frac{\exp(-\mu_n^2 \omega)}{\mu_n^2 (\mu_n^2 + \psi^2 + \psi)} \quad (18)$$

in which the μ_n are the positive roots of

$$\mu_n \tan(\mu_n) = \psi \quad (19)$$

In summary, the modelling employed numerical analysis by use of the method of finite differences producing values of moisture content as a function of time and specimen depth.

3 Experimental measurement of moisture content

Experiments were conducted by means of a novel moisture sensor to measure the DC resistance in different wood layers. The construction of the experiment is shown in Fig. 2.

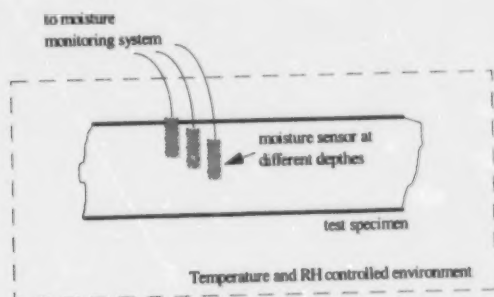


Fig. 2: A schematic of part of the testing system

A rectangular heartwood specimen of Beech was prepared (40×40×300 mm). This choice of species was dictated by the authors' own particular requirements. Holes, 10mm in diameter, were drilled into a number of Beech samples. The sensors were pushed into the holes and the holes were sealed with

silicon caulk to ensure that the outside atmosphere does not affect the buffer environment. Sensors were not inserted near obvious defects to ensure that the electrodes give more representative readings. Three sensors were installed in the same conditioning chamber at various relative humidity (RH) and temperature 20°C. The RH was maintained by appropriate saturated salt solution. The set-ups are described in Ahmet and Dai (1998).

Sensor construction was previously discussed fully in Dai and Ahmet (1998). This sensor employs wood buffers made from Beech with 8 mm in diameter and 25 mm long. Two pairs of silver painted brass screw-type electrodes are inserted into the wood buffer 12 mm apart along the buffer grain (see Fig. 3). A hole, 2.5 mm in diameter, allows the insertion of wires for connection to the electrodes. The 7-gauge shielded leads carry the signals over 4 m to the amplifiers. The isolating material (1 mm width) covers the electrodes to ensure that the electrodes do not touch the sample tested. The majority of the buffer is exposed inside the monitoring material in order that the buffer can reach equilibrium quickly.

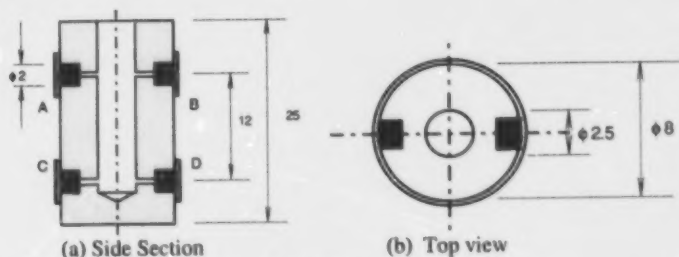


Fig. 3: Sensor model; dimension in mm

Combined with this specific sensor, a moisture monitoring system was developed with the characteristics of species compensation, anti-polarisation, accuracy improvement and noise reduction. The block diagram of the circuit is shown schematically in Fig. 4. A database pre-developed in TRADA (1998) was programmed into the system. Therefore, the system could give an accurate moisture reading compared with oven-dried date. The principles of the system and the sensor calibration can be found in Dai and Ahmet (1998). During moisture transmission processes in wood, moisture content based on DC electrical resistances in different layers have been continuously monitored.

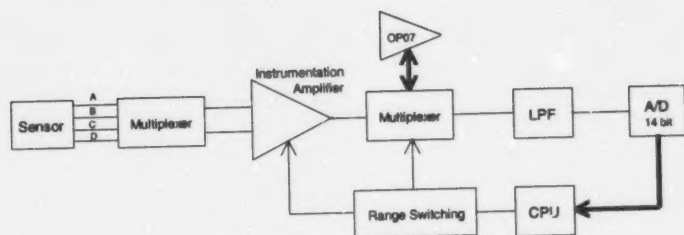


Fig. 4: Schematic diagram of the moisture measuring set up. (A/D: analog to digital converter; CPU: central processing unit; LPF: low pass filter)

4 Results and discussion

The predicted adsorption moisture contents using the numerical solutions method (section 2) and moisture content readings from the system discussed in section 3, are plotted against conditioning time in Fig. 5 for various depths below the wood surface. The "dry" samples were conditioned at 55% RH. Temperature was constant at 20°C for all experiments. For the numerical solutions, various values of the diffusion coefficient (D), based on curve fitting, were determined by equation (14). The dotted lines in Figures 5 and 6 have been extrapolated to low moisture content with caution since these go beyond the available experimental data. It is clear that there is good agreement between the predicted and measured moisture contents from the various depths employed.

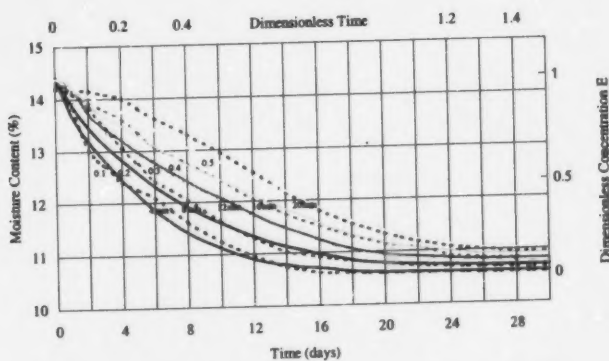


Fig. 5: Moisture content versus time for various wood depths (during adsorption). The solid lines result from numerical solution. The dotted lines show experiment data

In the case of desorption, the samples were pre-conditioned at RH=75% until they reached EMC; they were then conditioned down to RH=55%. Fig. 6 shows the predicted desorption moisture contents by numerical solution and moisture content readings from moisture monitoring system against conditioning time for various wood depths.

For both adsorption and desorption, the experimental data fluctuates around the EMC value after 20 days approximately. However the simulated data remained nearly "flat" close to EMC value. Sorption hysteresis is also observed, i.e., the final EMC of the adsorption curve being lower than that of the desorption curve. For comparison purposes, the prediction curve from analytical solution fell below the experimental

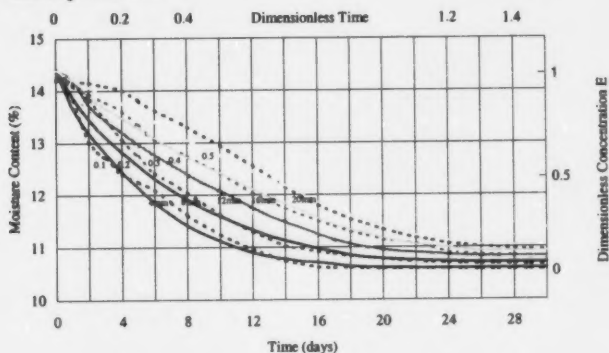


Fig. 6: Moisture content versus time for various wood depths (during desorption). The solid lines result from numerical solution. The dotted lines show experiment data

values. The difference between the analytical solutions from equation (18) and observed EMC became more substantial after 20 days. Analytical solutions have not been plotted in Fig. 5 to avoid clutter.

At the onset of conditioning, the specimen surfaces reach EMC instantaneously as a result of negligible mass transfer resistance at the wood/air interface. The moisture content at depth remains at the original value and there is an increase in moisture content just below the surface. As conditioning progresses, the moisture front moves towards the centre (opposite process for desorption) and the slope becomes smaller. At the same time, the moisture content in depth increases (decreases for desorption).

The agreement in trends between experimental data and numerical solutions is very reasonable when varying values of D are computed and employed as appropriate. Babiak (1995), Nakano (1995) and Hunter (1996) critically discussed the use of Fick's law for sorption by wood. These papers also stress the dependence of D on moisture content. (Although D depends on temperature also, this was kept constant in this study). Clearly, the diffusion coefficient is also a function of depth, that is, the diffusion coefficient of moisture transfer in wood specimen is dependent on its thickness. Avramidis (1987) quotes previous work to explain this phenomenon. They use the existence of a diffusion coefficient (D_i) due to instantaneous changes in the specimen because of the change of the

moisture content, and a delayed equilibrium diffusion coefficient (D_e). In the case of thick specimens the diffusion is so slow that the coefficient has no time to approach D_e closely, whereas diffusion near surface is so fast that the coefficient never changes appreciably from D_i . The diffusion coefficient is found to increase with the wood moisture content.

Although only transverse variations of moisture content have been investigated to date, this system will also be used to monitor the moisture transfer through the longitudinal direction of wood. In future work, a combination of both directions will also be trialed.

5 Conclusion

The model described in this paper, based on numerical analysis, was tested with sensors having the capability of monitoring the moisture content at different depths in wood. Continuous monitoring of moisture content has made it possible to overcome the drawbacks of various other investigations that are short of data or where erroneous data may have been employed. The model used here has reasonably described the process of adsorption and desorption of moisture in timber, where the transport of moisture follows the transverse direction of wood. There is potential to extend to moisture transfer along both the transverse and longitudinal directions of wood grain. The results from this model were compared with the experimental data and have shown good agreement over various conditions. The numerical simulation treats the boundary conditions more accurately than the analytical solutions especially where there are complex boundary conditions. The model makes it possible to attain the profiles of concentration of moisture developed throughout the sample.

It is expected to make further refinements which include the diffusion coefficient changes with depth of the specimen and temperature, and the moisture transfer along tangential, radial and longitudinal of the wood grain. These should enable the model to become a useful, versatile tool for studying moisture profiles and moisture related problems in building materials.

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67 THE RESIDUAL STRENGTH OF TIMBER DEGRADED BY WOODWORM INFESTATION

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Abstract

Evidence of beetle attack is found in roof and floor timbers of many old buildings. In Ireland widespread damage has been caused to numerous historic buildings by the Furniture Beetle (*anobium punctatum*). The timber is degraded both by the voids formed and possibly by the breakdown of the wood structure in the area attacked. It is relatively easy to stop the infestation but difficult to assess the structural soundness of the remaining timber. Removal and replacement of affected structural elements is usually difficult and expensive, particularly in buildings of historic interest. Current on-site assessment procedures are limited. A reliable technique to provide a quantitative assessment of the residual strength would be valuable for engineers and surveyors. This paper is concerned with the development of an on-site test of timber quality: a test which can be carried out on the surface and also at varying depths into the timber. It is based on a screw pull-out technique using a 'limpet' type load-measuring device. A correlation has been established between pull-out values and compressive strength test on laboratory samples for both sound and infested timber. Such a relationship is significant and could be used to assess whether post-strengthening work is needed.

Keywords: timber, structure, degradation, insects, field inspection, pull-out test, residual strength.

1 Introduction

Structural timbers are often degraded by fungi and insects. While fungal decay is extremely important in external timber its effect inside buildings tends to be localised. In contrast insect infestation is much more widespread within buildings and leads to a more general deterioration of the structure.

The most common wood-boring insect in Ireland is the Furniture Beetle (*anobium punctatum*) whose larvae, together with those of other beetles, are known as 'woodworm'. This species causes significant damage in buildings representing perhaps 80 - 85% of all insect infestation. When damage affects major structural elements such as roof trusses and floor beams removal and replacement of the timber can be expensive, particularly in historic buildings where retention of the structure, fabric and finishes is desirable.

It is relatively easy to stop infestation, but difficult to assess the structural soundness of the existing timber. Currently on-site assessment procedures are limited. A reliable technique to provide an quantitative on-site non-destructive assessment of the residual strength would be valuable for engineers and surveyors.

2 Background to assessment technique, preliminary studies

Clearly the voids created by the regular circular tunnels (diameter approx 1.5 mm) in the wood and the flight holes on the surfaces cause a reduction in strength. In addition it is thought that enzymes produced by the larvae may break down the cellulose in the cell walls adjacent to the tunnels. (Cartwright and Findlay 1958.)

The presence of an infestation is evident from the flight holes which appear on the surface of timber after completion of the first life-cycle from egg to adult beetle. The density (or proximity) of the tunnels in the underlying material should not be inferred from the surface appearance. The majority of the tunnels are produced by the larvae while the visible holes are made by the adult beetles when they emerge. Slicing and sanding of timber at different depths shows that the density of the internal holes can be much greater than that on the surface.

These studies were made on Baltic Redwood (*Pinus Sylvestris*) taken from roof trusses and floor beams of a flax scutching mill, some 200 years old, illustrated in Figure 1. In the floor situation the infestation was most severe on the top edge, in contact with the floor boards, diminishing in intensity towards the soffit of the beam: in the truss material the infestation was apparent on all faces.

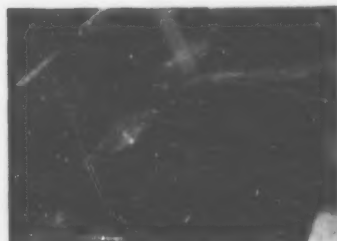


Fig. 1: Decayed mill roof truss section

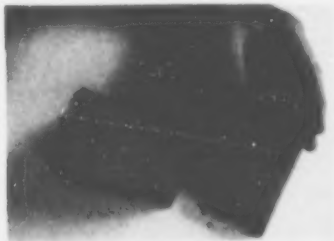


Fig.2: Test Probe in timber section

Slices were taken in both the radial and tangential directions relative to the growth rings. It became evident that the tunnels are formed in the less dense 'early wood' and generally run in the tangential direction between the adjacent rings of 'late wood'.

A test was envisaged in a direction at right angles to the longitudinal axis of a structural element which would generally exhibit a 'mixed sawn' pattern in cross section: i.e. the growth rings are not at right angles to either face or edge of the sawn section. From these studies it was clear that the testing technique should be applicable at different depths into the timber and at different angles to the grain, between radial and tangential directions.

The main objective of this programme was to develop an on-site test of timber quality: a test which could be carried out on the surface and also at varying depths into the timber. It is based on the force required to pull out a modified wood screw. By establishing a correlation between the pull-out values and the compressive strength of the timber, this will enable a more quantitative assessment of the infested timber to be made than that available by current methods.

It was also intended to investigate the effects of 'simulated' borings on 'old' and 'new' timber. The 'new' samples are to be taken from recently sawn European Redwood (*Pinus Sylvestris*) with approximately the same growth rate and strength as the 'old' timber. The results of this study, by comparison with naturally wormed timber, would enable possible additional degradation effects to be examined.

3 Experimental programme

3.1 Selection of test probe and measurement of pull-out force

An important requirement of the on-site test was that damage to the surface of the timber element should be kept to a minimum. This is important for sampling in the areas of sound timber more so than in the areas of severe infestation as this latter region is likely to be subject to remedial works in any case. A relatively small diameter screw of 5 mm was selected. In order to sample through the depth of the member the length of the screw needed to be at least 100 mm. To enable a comparative profile of strengths at a range of depths to be obtained the length of the threaded portion of the screw was reduced by machining off all but 20 mm of the thread at the tip. Figure 2 shows the screw in place in a sectioned piece of timber. It can be seen that a clearance hole has been drilled to ensure that the test screw samples timber only at that depth.

A device to measure the on-site compressive strength of concrete has been developed in the Department of Civil Engineering, Queen's University, known as the 'Limpet'. (Long and Murray 1994) This consists of a small hand-held loading device, incorporating a load cell, which measures the pull-out capacity of a probe in the form of a bonded-on metal disc. The Limpet was used in this timber testing programme by substituting the test screw for the surface disc as shown in Figure 3.

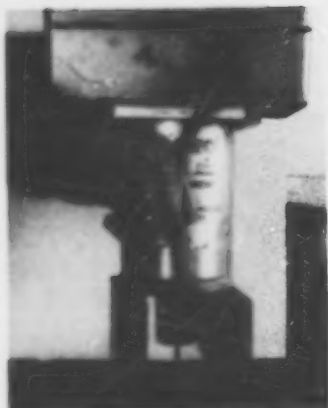


Fig. 3: Limpet pull-out device

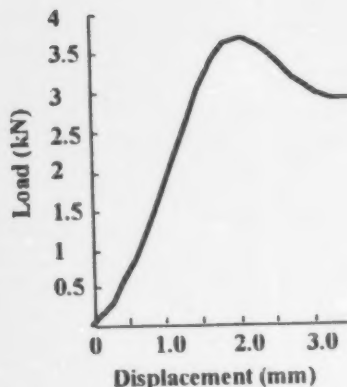


Fig. 4: Tensometer plot

A spherical connection was used between the head of the screw and the load cell to ensure that only axial load was applied.

As a check on the accuracy of the pull-out values given by the Limpet device, some samples were also tested using a standard tensometer machine. Good comparison was achieved between both devices. A typical plot from the tensometer is shown in Figure 4, which illustrates the nature of the load-displacement behaviour.

3.2 Timber samples and test procedure

European Redwood, of recent origin was used for the control samples in the laboratory. It continues to be one of the most commonly used softwoods in the construction industry.

A preliminary investigation was carried out to examine the effect of the screw's angle of entry into the timber. A fully threaded screw, 100 mm long, was inserted into a sizeable sample of new clear timber. The depth of the pull-out testing ranged from 10mm to 60mm in 5mm intervals in the three principal directions: radial to the grain, tangential to the grain and parallel to the grain.

For the main testing programme, decayed timber from the mill trusses was used for the pull out tests with the partially threaded screw. These were done at different depths in two ways: first by drilling clearance holes to the required sampling depth as shown in Figure 5(a); then, in order to check for any possible constraining effect of timber above the sampling level, tests were also carried out on a 'stepped' specimen in which horizontal slices were cut to expose five different sampling platforms as shown in Figure 5 (b).

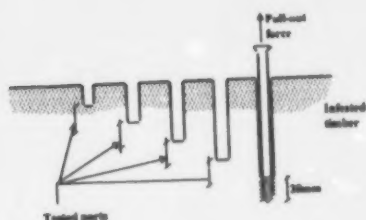


Fig. 5 (a): Pull-out testing with clearance holes

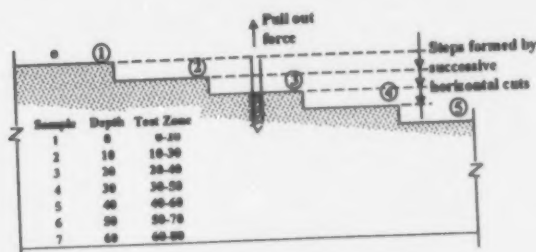


Fig.5 (b): Pull-out testing on stepped specimen

This programme of testing using the partially threaded screw on old wormed timber from the mill trusses can best be considered in the following sets:

Set A: depth tests – clearance holes – specimen 1

Set B: depth tests – stepped levels – specimen 1

Set C: depth tests – stepped levels – specimen 2

Following these sets of pull-out tests, samples 20 x 20 x 60mm were cut out from each platform, close to the test hole of the pull-out screws. These were then tested in compression according to BS 373: methods of testing small clear specimens of timber (British Standards Institution 1986). A typical compression test is shown in Figure 6.

4 Test results and evaluation

The results of the preliminary fully threaded screw tests into clear timber are shown in Figure 7. From the graph it can be seen that the weakest pull-out forces occur, as expected, when the screw is inserted parallel to the grain. Both the tangential and radial directions gave very similar results. This would appear to be due to the fact that the screws are penetrating approximately the same length of early wood and late wood in both radial and tangential directions. A linear relationship is apparent between pull-out force and fully threaded screw depth for all three directions.



Fig. 6: Typical compression test

Table 1 presents the results of the Set A and Set B tests using the partially threaded screw on a specimen of wormed timber. For both Set A, using clearance holes and Set B, using the alternative depth sampling by slicing the specimen to form steps, it can be seen that very low pull-out values are measured near the surface and then increase with depth. There is good agreement between the results of Set A and Set B. This shows that the containment effect is not significant. A comparison with 'new' clear redwood pull-out capacities is also given. These tests show, as expected, very little variation in pull-out force with hole depth, again confirming that the containment effect is not influencing the results. Results of the compression testing of samples cut from the decayed timber at various positions close to test holes are listed.

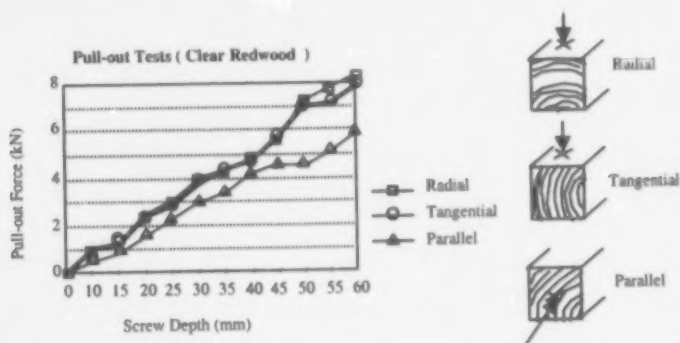


Fig. 7: Variation of pull-out force with angle of entry

Table 1: Summary of Test Results (Sets A & B)

Depth of clearance hole mm	Depth of sampling zone mm	Pull-out Force kN		'New' clear timber	Compression capacity kN	
		Wormed timber			Wormed timber	'New' clear timber
		Set A: Clearance holes	Set B: Stepped holes		Clearance hole	Set A & Set B
0 (surface)	0 - 20	0.08	0.47	2.07	1.73	17.79
10	10 - 30	0.40	-	2.09	-	
20	20 - 40	0.84	1.21	1.95	6.42	
30	30 - 50	1.29	-	1.78	-	
40	40 - 60	1.53	1.60	1.84	8.49	
50	50 - 70	1.51	-	1.99		
60	60 - 80	-	1.43		11.52	

Note: All compression capacities based on an average of 3 samples.

Table 2 summarises the findings of Set C tests which included consideration of the variation of insect boring intensity with depth. It shows pull-out values, compression capacity and hole intensity. Again, a steady increase in the pull-out value and compression capacity with depth can be observed. In addition there is a correlation of both values with hole intensity: Figure 8 illustrates the relationship with pull-out force. The hole intensity was based on a count within an area 20mm x 40 mm, taken on a radial plane in the timber piece, since the tunnels generally lie between the concentric grown rings. Because most samples exhibited a 'mixed sawn' pattern, it was necessary to count the holes on a projection of the 20 x 40 mm area onto the stepped platform.

Figure 9 plots the relationship between pull-out force and compression capacity shown by the results of Set A, Set B and Set C tests. Values from Sets A and B were averaged (tests carried out on specimen 1): Set C values are identified separately since tests were performed on timber taken from another truss.

The final phase of the investigation was designed, as mentioned in the introduction, to examine the effects of 'simulated' borings on old and new timber. A 1.5mm diameter drill was used to produce both regular and random patterns, (the latter produced by random number generation) and different intensities on the 20 x 40 mm plane.

The results of this programme are not included in the paper. They indicate, at the present stage of the work, that there appears to be no degrading effect beyond that produced by the holes, provided that the infestation is not associated with fungal decay.

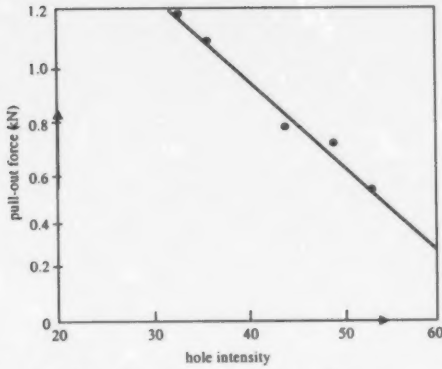


Fig. 8 : Variation of pull-out force with hole intensity

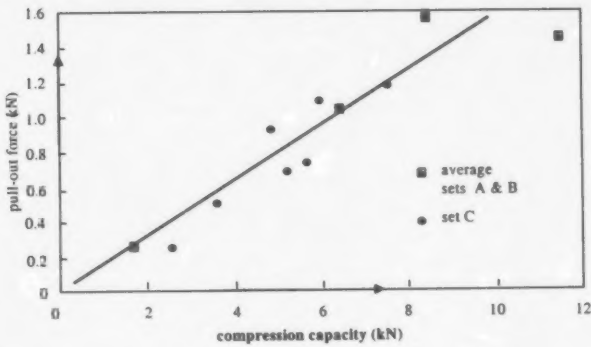


Fig. 9: Variation of pull-out force with compression capacity

Table 2: Summary of test results (set C)

Depth of sampling Zone mm	Hole Intensity	Pull-out Force KN	Compression capacity KN
0 - 20	+	0.25	2.6
10 - 30	53	0.49	3.6
20 - 40	49	0.67	5.2
30 - 50	44	0.73	5.7
40 - 60	40	0.92	4.9
50 - 70	36	1.07	6.0
60 - 80	33	1.17	7.6

+ not counted.

5 Conclusions

The intensity of infestation by Furniture Beetle in a piece of timber cannot be inferred from an examination of the exposed surfaces. Cross-cutting and longitudinal sawing demonstrates that the intensity of internal borings can be much greater than that showing externally. Surface examination is therefore not a reliable guide to structural soundness and there is a clear need for a quantitative non-destructive testing technique which can sample within the timber element.

The pull-out force values measured by the experimental probe are essentially the same for probe axis directions both radial and tangential to the growth rings. For practical on-site application this is important since most timber elements are likely to show a 'mixed' sawn pattern when viewed in cross-section. Since the force value parallel to the grain is different, the probe should always be inserted at

right angles to the surfaces, in an area clear of knots. The influence of grain slope should not be significant, even considering the slope limitations which apply to the lower grades in contemporary practice.

The probing technique, using clearance holes to sample at depth, gives reliable results. There appears to be no 'containment' or 'arching' effect due to timber above the flight of threads. This is consistent with the direct relationship between pull-out force and thread length for the fully-threaded probe.

The examination of the effects of 'simulated' boring in old and new timber has not been conclusive. At the present stage of investigation there appears to be little or no degrading effect on compressive strength in addition to that produced by the tunnel voids.

There is an inverse relationship between hole intensity and pull-out force. The hole count should be made on the plane radial to the growth rings since the tunnels generally run tangentially to the rings.

The pull-out force values can be correlated with compressive strength. The technique developed is a viable test of timber quality. The scope of the investigation has been limited to one species, from one structure, although samples were cut from different elements. Compressive strength has been used as the indicator of residual strength. Work is progressing to include impact strength.

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68 PROTECTION OF WOOD BY DESIGN

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Abstract

Environmental concerns call for limited use of chemically treated wood for exterior wall cladding. Instead, durability should be guaranteed by using *protection by design* or by using wood of improved quality. SBI participated in a Nordic project concerning wood protection by design and carried out a number of tests in the laboratory. The durability of timber claddings was evaluated by testing wood samples exposed to artificial ageing from cyclic influences from sun, rain, frost and thaw in *The Four Seasons*, a climate simulator. Six different tree species as well as various surface treatments and orientations etc. were tested. The two species that did best were larch and thuja. In *The Driving Rain Chamber* the protecting effect of different types of painted drip caps was tested. The protection right under the drip cap increases with the depth of the drip cap. Drip caps with inclinations of 1:5 and 1:4 provided the best protection and a notch on the underside of the drip cap increased the protection.

Keywords: Durability, pressure impregnated wood, wall cladding, protection by design, climate simulator, drip caps

1 Introduction

Environmental concerns call for limited the use of chemically treated wood for exterior wall cladding. Instead, durability should be guaranteed by using *protection by design* or by using wood of improved quality. SBI participated in a Nordic project concerning wood protection by design and carried out a number of tests in the laboratory. The present paper describes a part of the test results from this project. A

more detailed description of the project can be found elsewhere in these proceedings (Vestergaard 1999) and in (Vestergaard et al. 1997).

2 Materials and methods

2.1 Testing of wood materials

Wood test specimens measuring 19 by 85 by 361 mm were tested by artificial ageing. In total, 288 specimens were artificially aged simultaneously. The artificial ageing took place in a test facility that has been designed for ageing of test specimens or small building components. The test facility, which is known under the name *The Four Seasons*, is shown in Fig. 1. A more detailed description of *The Four Seasons* is found in (Nielsen 1995).

The 8-hour climate cycle used in *The Four Seasons* was composed of 2-hour periods of artificial sunlight, rain, frost and laboratory climate. The artificial sunlight raised the temperature of the test specimens to 45 °C in average and had an acceleration factor for UVB radiation of 7.5. In the rain period 15 l water/m²·h were sprayed onto the surface of the test specimens for 30 min during the first 2880 hours, 15 l water/m²·h for 1 hour during the following 600 hours and 25 l water/m²·h for 1 hour during the last 400 hours of exposure. In the frost period, the test specimens were cooled to 5 °C in average. The average condition of the air in the laboratory during the test period was 25 °C and 35 % RH. The artificial ageing lasted 3880 hours, equalling 485 cycles. The parameters tested were:

- Wood species (oak (heartwood)/larch (heartwood)/Douglas fir (heartwood)/ thuja (heartwood)/pine (heartwood)/pine (sapwood)/spruce (heartwood)/spruce (sapwood))
- Orientation/notching on back (heartwood exposed/sapwood exposed/heartwood exposed and notched/sapwood exposed and notched)
- End-grain sealing (3 x paint/1 x paint/1 x glue/none)
- Surface treatment (pine (none)/spruce (wood oil)/spruce (linseed oil)/oak (wood oil)/oak (linseed oil)/thuja (wood oil)/thuja (linseed oil))
- Surface character (sawn/planed (long shavings)/cutter planed/painted with ICP reference paint)
- Fastening (dowels/screws in predrilled holes/screws/nails/gunned nails)
- Vertical/horizontal exposure (pine horizontal/pine vertical/spruce horizontal/spruce vertical)

The following measurements were made on all test specimens before, five times during and after exposure:

- Moisture content (determined gravimetrically)
- Cupping
- Cracking

The temperatures and moisture content calculated from AC resistance were datalogged every 30 minutes throughout the whole period of exposure in 48 test specimens. Moreover, photographs were taken before, tree times during and after

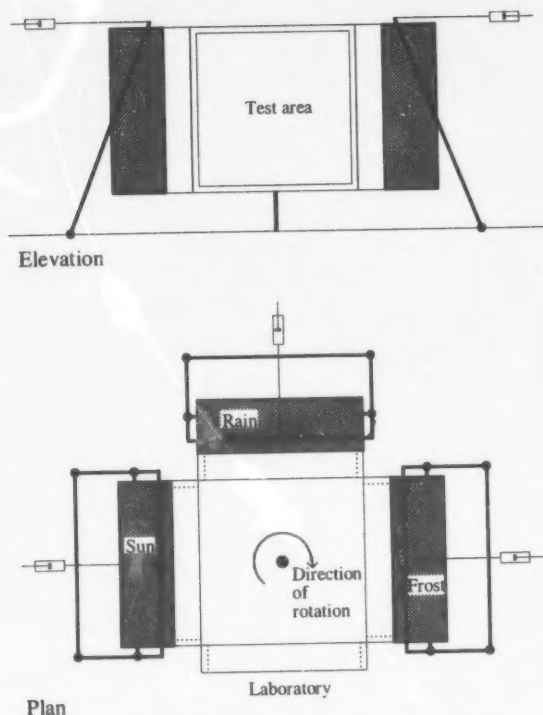


Fig. 1: Elevation and plan of *The Four Seasons* climate simulator where vertically mounted test specimens are exposed to a climate cycle composed of artificial sunlight, rain, frost and laboratory climate. The test area measures 1.6 m by 1.6 m

exposure. Finally, microbiological examination and CT scanning were carried out on a few selected test specimens.

2.2 Drip cap testing

In the *Driving Rain Chamber* the protecting effect of different types of painted drip caps was tested. The drip caps were made of pinewood and were surface treated with fungicidal oil and painted twice with plastic alkyd paint. The parameters of the drip cap test were:

- drip cap depth (45/70/95 mm)
- drip cap slope (1:10/1:5/1:4/1:3)
- type of notching on the underside (no notch/circular notch/sharp-edged notch).

Drip caps with a length of 1 m were mounted on a pane of glass, see Fig. 2. A continuous water flow of 206 l/h·m was supplied at the top of the glass of pane, from a pipe with holes. This high flow rate was chosen in order to obtain repeatability. A gutter with a width of 70 cm and a narrow opening was in turn placed at the pane of glass 25, 55 and 85 cm below the underside of the drip cap. The water captured by the gutter was led through a hose into a bucket that was weighed after 5 minutes of accumulation

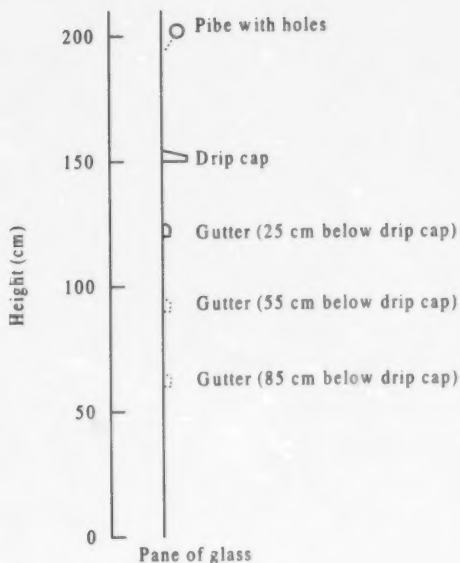


Fig. 2: Set-up for drip cap testing

3 Test Results

3.1 Wood materials

Part of the results from the testing in *The Four Seasons* is shown in Figs. 3 and

4. Pine sapwood was used throughout the test unless something else is indicated.

The effect of wood species on moisture content determined gravimetrically and cupping is shown at the top of Fig. 3.

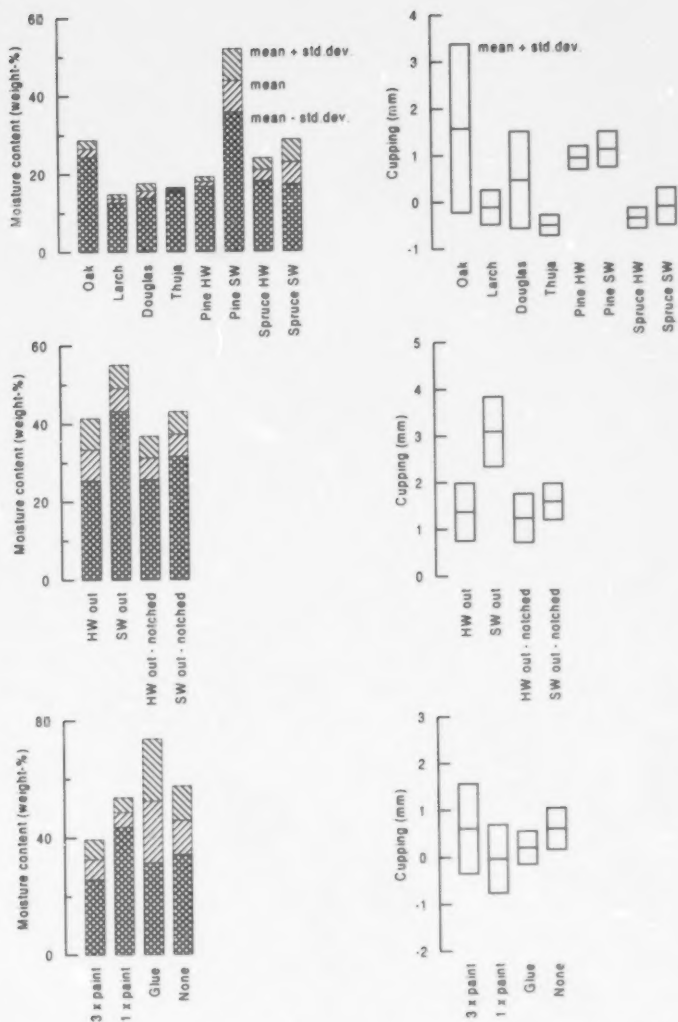


Fig 3: Moisture content and cupping as a function of wood species (top), orientation/notching (middle) and end-grain sealing (bottom). HW and SW denote hardwood and sapwood respectively

When observing the tendency to cupping, it should be borne in mind that a low mean value does not necessarily signify dimensional stability of the wood. This is a consequence of the fact that high positive values of cupping may be cancelled out by equally large negative values of cupping. Consequently, the mean value of cupping should only be observed in connection with the standard deviation.

Unfortunately, due to an error in the design of the test rig, no weighing or cupping results were obtained from the part of the test aimed at investigating the effect of horizontal versus vertical exposure.

No significant differences were observed between the cupping of test specimens joined with different nailing methods.

Visually observed, the formation of cracks could be arranged in the following order of increasing tendency to cracking

- Thuja
- Larch
- Spruce heartwood
- Oak and pine heartwood
- Spruce sapwood and Douglas
- Pine sapwood.

The microbiological examination of three test specimens showed traces of mould fungi. This was unexpected because it was anticipated that the high temperatures in the "summer" part of *The Four Seasons* would destroy the viability of fungal spores.

2.3 Drip caps

Results from the drip cap testing are shown in Figs. 5-8 in terms of effectiveness. The error bars in Figs. 5-8 denote the sample standard deviation. The effectiveness E (-), at a given distance x (m) below a drip cap, is defined as

$$E = \frac{g_0 - g(x)}{g_0}$$

where g_0 (kg/m s) is the linear flow rate above the drip cap and $g(x)$ (kg/m s) is the linear flow rate below the drip cap.

3 Discussion

3.1 Wood materials

As expected, the moisture content of pine sapwood was substantially larger than that of the other wood species tested, see Fig. 3 (top). Pine heartwood, Larch, Douglas and thuja had acceptable moisture content after exposure. As regards cupping oak had the poorest performance. Furthermore, Douglas spruce exhibited a large cupping standard deviation.

Proper orientation of pine sapwood, ie leaving the heartwood side exposed, is

essential to reduce cupping. If notching on the backside is applied, cupping of pine sapwood with the sapwood side exposed can be controlled and kept at the same order of magnitude as when the heartwood side is exposed, see Fig. 3 (middle).

Three layers of paint reduced the water content of the test specimens significantly as compared to the moisture content of those without end-grain sealing, see Fig. 4.

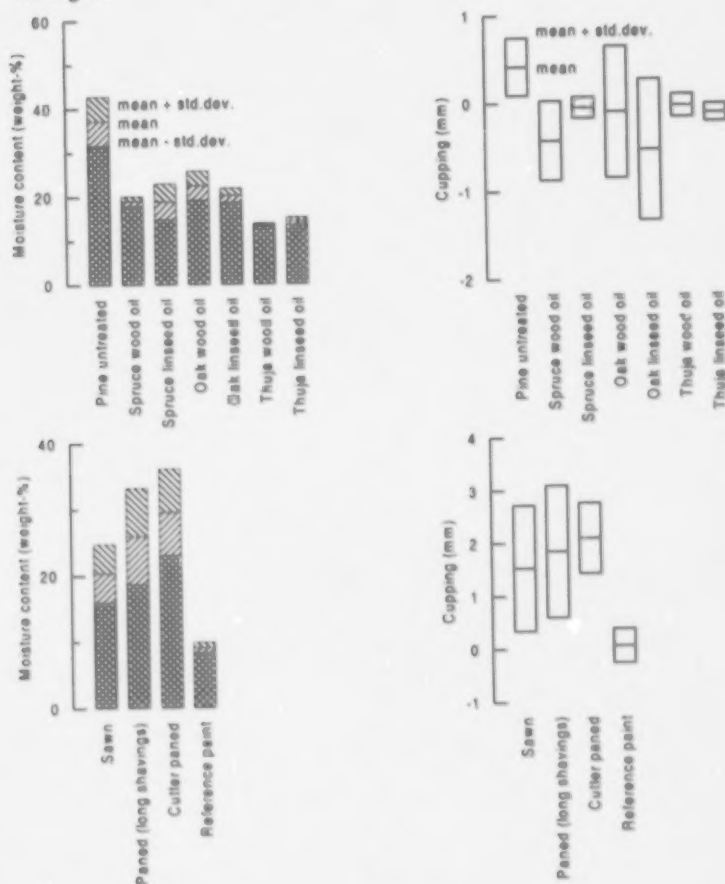


Fig. 4: Moisture content and cupping as a function of surface treatment (top) and surface character (bottom)

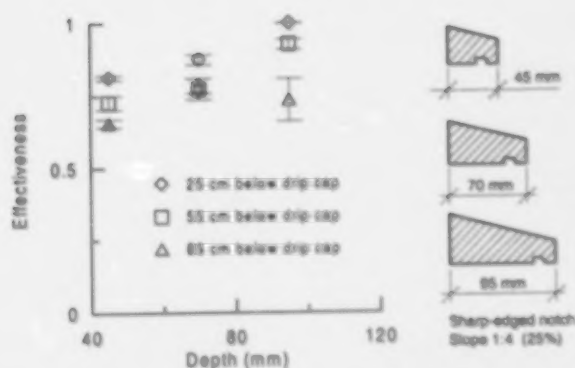


Fig. 5: Drip cap effectiveness 25, 55 and 85 cm below drip cap as a function of drip cap depth. Sharp-edged notches with 1:4 slope

Surface treatment with wood oil and linseed oil performed almost equally well regarding moisture content and cupping, see Fig. 3. When compared with non-treated specimens, surface treatment did reduce moisture content and cupping, compare Figs. 3 and 4.

No significant differences were observed between the pine sapwood test specimens with sawn, paned or cutter paned surfaces neither in terms of moisture content nor in terms of cupping, see Fig. 4 (bottom). These three types of surface character performed poorer than the painted reference specimens.

3.2 Drip caps

The performance of the various drip cap designs in this section is discussed well knowing that the action of wind forces might have changed the results considerably.

Unsurprisingly, the protection right under the drip cap increases with the depth of the drip cap. Conversely, this positive effect diminishes as the distance below the drip cap increases, see Fig. 5. Good performance is seen to be obtained up to 25 cm below the 95 mm deep drip cap.

The effect of notching was dependent on notch shape/presence as well as on drip cap slope. With a 1:3 slope and a depth of 95 mm, drip caps with a circular notch were quite effective. With a 1:10 slope and a depth of 95 mm, drip caps with a sharp-edged notch had a slightly better protective effect than drip caps without a notch or with a circular notch.

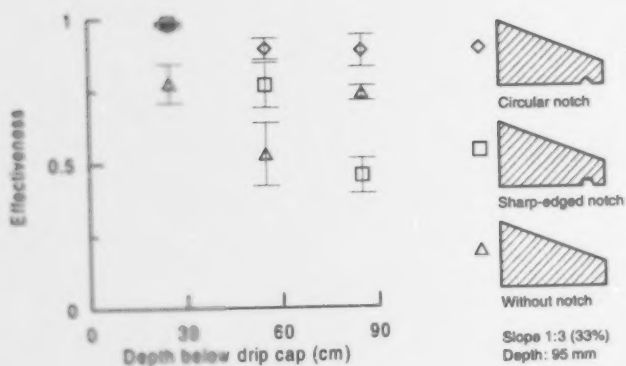


Fig. 6: Drip cap effectiveness for various notch shapes as a function of depth below drip cap. Slope 1:3 and 95 mm drip cap depth

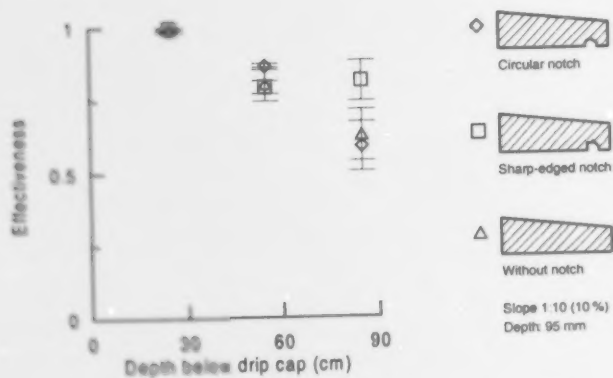


Fig. 7: Drip cap effectiveness for various notch shapes as a function of depth below drip cap. Slope 1:10 and 95 mm drip cap depth

The overall impression from the results in Figs. 6-7 is that notches tend to make a difference, but not as much as expected. However, it should be noted that the tests were made under ideal laboratory conditions and not on a real building. In the laboratory the drip cap underside was always horizontal, which might not be the case in a real building. A wall on a real building with a drip cap whose underside slopes against the wall will almost certainly benefit from a notch. Irrespective of slope all the 95 mm deep drip caps with a sharp-edged notch provided good protection up to 25 cm below the drip cap, see Fig. 8. Further below the underside, drip caps with inclinations of 1:5 and 1:4 were most effective.

4 Conclusions

Larch, Douglas, thuja and pine heartwood exhibited relatively moderate moisture uptake. As regards cupping larch, thuja and spruce heartwood also had small mean values and small standard deviations.

Notching on the backside of pine sapwood was shown to decrease cupping when the sapwood side was exposed to a varying climate.

Of the methods investigated, 3 layers of paint provided the most effective end-grain sealing.

As expected, drip cap effectiveness improved with drip cap depth. Drip caps with inclinations of 1:5 and 1:4 provided the best protection and a groove on the underside of the drip cap increased the protection.

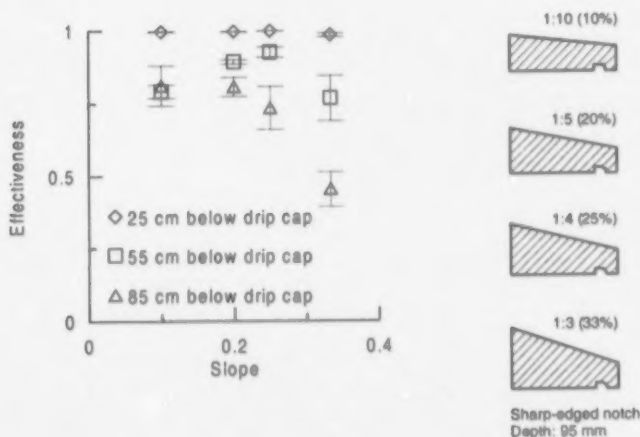


Fig. 8: Drip cap effectiveness 25, 55 and 55 cm below drip cap as a function of slope. Sharp-edged notches. 95 mm drip cap depth

5 Acknowledgements

The financial support for this project from Nordic Wood programme under the Nordic Industrial Fund, the Danish EPA and the EU LIFE programme is gratefully acknowledged. The selection of wood species and the experimental design for *The Four Seasons* was carried out by C. J. Flarup formerly Danish Technological Institute, presently McKinsey & Co and M. Vestergaard, Danish Technological Institute. Most of the measurements were made by L. P. Utrup, Danish Building Research Institute.

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69 DESIGNING FOR DURABLE WOOD CONSTRUCTION: THE 4 DS

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Abstract

While there are examples of ancient wood buildings which have stood the test of time, there are also examples of new buildings which have suffered deterioration problems within a few years of construction. Changes in materials, design and construction methods have been suggested as the culprits and there has been a tendency to look for a magic bullet to solve the problem. Recent work shows that a number of factors have combined to cause these problems. The solution has to be equally multifaceted. Although wood is a very versatile building material, designers need to bear in mind its biological origin when using it in construction. Moisture management is the key and, in the temperate rainforest climate of coastal British Columbia, controlling rain penetration is critical. Such control requires defense in depth which can be summarised as 4 Ds: Deflection, Drainage, Drying and Durable materials. In combination, these may provide sufficient capacity to balance the rainfall load. However, it is necessary to provide over-capacity to allow for imperfect design and construction and for aging. Durable wood construction therefore requires 4D thinking in the sense of four-dimensions: three-dimensional detailing, and the effect of time, the fourth dimension, on long-term performance.

Keywords: Durable, wood, construction, deflection, drainage, drying, durability.

1 Introduction

This paper describes an approach to defense in depth against rain penetration, particularly for multi-occupancy residential wood-frame buildings in a temperate cool wet climate. In Vancouver, there are many examples of wood frame buildings which are 60 to 80 years old with no signs of moisture problems. Europe and Japan have examples centuries old. Recently, a rash of problems with water leaks in multi-occupancy buildings in Southwestern British Columbia has raised awareness within the construction industry of the need to improve the entire process of design, construction and building operation.

In 1995, Canada Mortgage and Housing Corporation (CMHC) set up the Building Envelope Research Consortium (BERC) to research technical solutions to the problem. A field survey (Morrison Hershfield 1996) confirmed that the major problem was rain penetration, rather than construction moisture, or air leakage leading to condensation. It highlighted a combination of poor, or absent, design details and poor construction practice, not poor quality materials or lack of maintenance, as the major areas of concern. Rain entered at penetrations, not through the face of the cladding. Problems occurred on buildings with vinyl and wood siding as well as stucco-clad walls, but stucco was the dominant cladding type used during this period. Stucco-clad walls did have more extensive symptoms suggesting that stucco may have impeded drying of these walls. The survey pointed to window-surrounds, saddle flashings, balconies and walkway membranes as key areas where rain was penetrating the envelope. Buildings with simpler designs performed better. They had fewer water-trapping features and fewer penetrations where errors in design and/or construction could lead to water ingress. East- and South-Facing walls had more problems than North- or West-facing walls, probably because wind-blown rain normally comes from the East and South-East in the Vancouver area. The presence and width of overhangs was correlated with fewer problems. In general, these conclusions, based on data, supported those based on observation and experience. They formed the basis for commencing the localized Best Practice Guide (BPG).

The BPG was funded by CMHC and published in basic form in the fall of 1998. It contains specific information for wood frame apartment design and construction in the coastal climate area of BC. The document is over 200 pages, contains 29 detail drawings and 6 animated computer files showing sequential application of components (Morrison Hershfield and RDH Building Engineering Ltd. 1998).

During the discussions of the BPG steering committee it became obvious that while the complex nature of the problem required a complex set of solutions, a simple set of basic principles could get the core message across to the construction industry. It was found that the basic principles for management of rain penetration could be described in terms of four words beginning with the letter D.

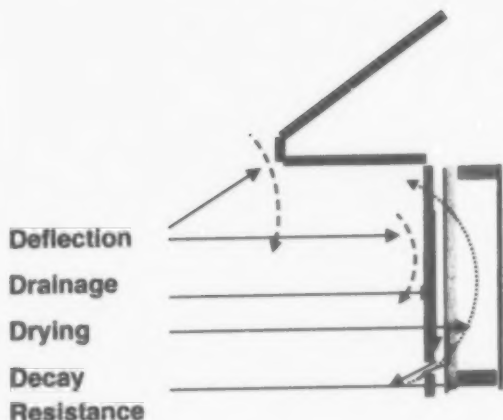


Fig. 1: The four D's of wall design

2 The Four Ds

Deflection: Wherever possible rain should be deflected so that it fails to hit the wall. Wind-blown rain that does impact the wall, should be deflected to drip away from it. Roof overhangs are the primary rain deflectors. Siding profiles and flashing are two design elements which can promote deflection at the wall surface. Pressure-equalized cavities can also be considered as deflection mechanisms since they eliminate some of the forces which can promote ingress of water through the cladding (Morrison Hershfield 1994). Deflection is the first line of defense for the wall system and this is estimated to be capable of dealing with perhaps >90% of rain incident on a building in the Vancouver area, given a properly designed deflection system. The rainfall load on the walls is then reduced to <10%. Sufficient is known about this phenomenon to design buildings to provide deflection.

Drainage: If water penetrates the cladding it should be drained out as quickly as possible. Research over the years has shown that face-sealed walls will only work in very special circumstances and therefore the majority of buildings should use a concealed barrier or drained cavity system (Morrison Hershfield 1992, 1994). Vinyl siding and wood siding over battens can provide this drainage path. There is considerable discussion as to whether drainage is possible between stucco and the

building paper, using one or two layers of paper. Due to doubts as to the effectiveness of this system, the City of Vancouver has mandated a 19mm cavity using battens behind stucco. Recently, outside the City of Vancouver some construction has incorporated stucco systems using 4-8mm geomats (drainage matting) intended to provide enhanced drainage capacity. Most of the detailing which was recommended based on the Atlantic moisture studies (Canada Mortgage and Housing Corporation 1994) was of the drained-cavity type, however, all of this work was on wood or vinyl siding, rather than stucco. Nevertheless, there is a reasonable knowledge base on drainage.

A properly detailed drainage system can theoretically deal with the remaining 10% of water which might pass the outer cladding barrier. Theoretically this reduces the load to < 1% which is a reasonable target. It is highly likely that no wall construction is perfect. Liquid moisture will find a way into the wall and vapor pressure reversals will occur driving moisture into the wall, therefore a third line of defense is required.

Drying: In the event that moisture penetrates or circumvents the cladding and the moisture barrier (and to deal with initial construction moisture) walls should be designed to dry to the outside. Research by Forintek shows that the Vancouver coastal climate will allow protected and ventilated lumber to reach an equilibrium moisture content of <19% (Forintek unpublished data). Thus, wood frame drying to the exterior may be possible in this climate. However, systems of wall construction used in recent years may not provide adequate drying rates in this climate, even if the ingress of water can be minimized by proper detailing. Previous research provides analysis tools which predict that vapour migration is insufficient in itself to dry bulk moisture from walls in short periods in Vancouver (National Research Council, unpublished data). Once a piece of wood is infected with the spores of decay fungi, decay advances at a rate dependent on the temperature and moisture levels in the wood. Since temperatures in coastal BC do not fall low enough over the winter to retard fungal growth until moisture levels decline below the problem level in summer, moisture levels must be reduced relatively quickly. How quickly is a matter of some debate, but it is generally thought that if the moisture level stays in the +30% range, decay fungi may become established in as little as 3 months. A project designed to more accurately define the threshold time at a range of moisture contents and temperatures which will lead to mould and decay is underway.

Decay Resistance: This could also be termed moisture tolerance or safe storage of water. In its natural state wood has a built-in moisture tolerance. The perishable sapwood of softwoods has the ability to tolerate moisture contents up to 20% on a continuous basis without suffering biodegradation. Higher moisture contents can also be tolerated for shorter time periods (Viitanen 1997a, 1997b). The heartwood of moderately durable species, such as Douglas fir, may be able to tolerate moisture contents up to 23%. Durable wood species, such as western red cedar may be able to tolerate moisture contents up to 35%. As a result, wood systems have a higher capability than steel systems to safely store small amounts of water. A 1.2m x 4.9m (4ft x 8ft) section of wood frame wall can tolerate around 5kg water (increasing from

12 to 20% mc) without suffering from decay or mould growth. In a steel frame wall, 5kg water could be trapped as liquid in the track leading to corrosion. Where the first three Ds can not reliably maintain wood components continuously below 20% moisture content (or higher for a only few days at a time) the decay resistance of the wood must be enhanced. This is typically done via pressure preservative treatment. The type and level of treatment required will depend primarily on the severity of the moisture hazard and the types of wood-destroying organisms in the local area.

Employing these four Ds, could help those involved in design and construction to ensure the durability of wood buildings.

3 Load/capacity balance throughout the building life cycle

The development of a Best Practice for Wall Design by the BERC necessitated the consideration of the evolution of the wall from the initiation of the project by the developer through to the end of the mortgage period and eventually the end of the useful life of the building. The life of the building can be considered to comprise seven stages:

- 1 Initial concept and preliminary design response to market.
- 2 Design Development.
- 3 Construction Documents
- 4 Construction
- 5 Initial occupation to five years.
- 6 Five years to mortgage period of 25-years
- 7 Twenty-five years to life expectancy of 80 years.

Stages 5, 6 and 7 were separated to accommodate the different effects of aging over the short and long term. Planning for success in the functioning of the building envelope has to consider the activities impacting on the envelope at each stage in the building's life. As with structural performance, there are loads which will come into play at each stage, all working to cause the structure to fail. In the case of structural design, these are gravity loads, occupancy loads, wind loads, seismic loads, and thermal loads. All have to be accounted for and additional levels of capacity built into the design of the structure to deal with events based on their likelihood of occurrence. In the case of the building envelope design for durability in a rainforest climate, rainfall could be considered as a load impacting on moisture management performance. Additional loads include construction moisture, plumbing leaks, soil moisture, and humidity resulting from occupant activity, but these are not the subject of this paper. For the purpose of rain penetration control, the load can be considered as the total amount of water falling on a point in space annually. In the case of **Deflection**, capacity is the degree to which that rain is prevented from falling onto the wall or water on the wall is encouraged to drip away from the wall. In the case of **Drainage** and **Drying** capacity it is moisture removal in liquid and vapour form, and in the case of **Decay resistance**, it is safe storage capacity without component failure.

Each decision in the design and each trade involved in construction affects the load on, or the capacity of, the system. The load must be accommodated in the design of the envelope with sufficient capacity to prevent failure.

4 The Acknowledgement of imperfection in the design, construction and operation of walls

Failure of the building envelope can be defined as "failure to prevent build-up and retention of moisture greater than the safe storage capacity of the wall system", or "failure to prevent ingress of water into the building interior sufficient to cause substantial damage to fixtures and fittings".

The causes of failure are as numerous as the architectural details and the number of parties involved in the design and construction of the building exterior. It becomes a complicated task to trace all the factors. The underlying problem is easier to identify. The concept, design, construction, and operation stages of wood frame envelopes is based on an assumed degree of perfection which is impossible to achieve. The reality is that all these stages of the life of the building have a probability of perfection less than 1.0 potentially contributing to failure unless this imperfection is compensated for in the original design.

The total capacity at each point is the sum of the capacities for deflection, drainage, drying and decay resistance (Figure 2). The height of the left-hand side of the column for each stage in the life cycle represents the intended capacity as a percentage of the load. The height of the right hand side of the column at each stage indicates the effective capacity (Figure 2).

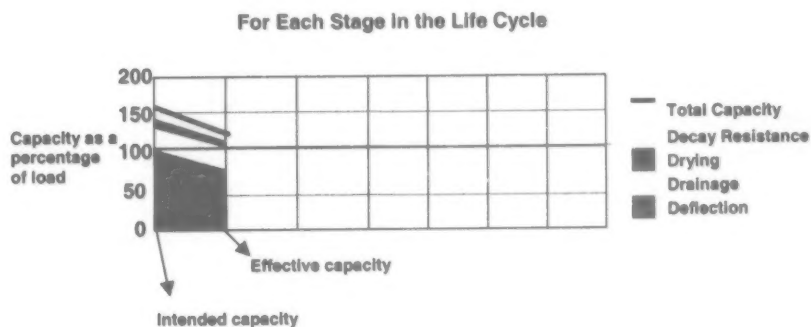


Fig. 2: Intended capacity vs effective capacity

In the case of stages 1 to 4, this takes into account the probability of perfection and the impact of less than perfect design or construction in reducing capacity. In the case of stages 3 to 7, this takes into account aging and weathering effects in reducing the capacity of moisture management systems.

5 Using load/capacity balance and probability of perfection to assist decision making throughout the life of the building

In Figures 3 to 6, the block of shading should be thought of as a series of histogram columns, one for each stage of the life cycle. The slope at the top of each column comprises, in reality, a sequence of small steps. The degree to which the capacity exceed the load in Figures 3 to 6 is merely notional and is intended for illustrative purposes only.

5.1 Stage 1: Initial concept and preliminary design response to market

The marketing strategy for the target consumer group determines or suggests a look for the project. Recently in the Vancouver Market this has been a California or Arizona look. Typically this look has complex architectural features, flat roofs, parapet walls, exterior walkways and columns, balconies for every apartment, and stucco cladding. The absence of overhangs is also typical, but this is due, in part, to Vancouver zoning bylaws which include some overhangs in the built area.

Stage 1 is a critical time in the design of the envelope. If the developer chooses a "heritage" look with generous overhangs then the loads on the wall systems will be low and the total capacity, even with a lower cost wall will be high.

Figure 3 illustrates a theoretical condition for a true heritage stucco-clad building, many of which are still performing well in Vancouver after 60 years. It has been suggested that these had a greater inherent level of deflection, drainage, drying and durability than modern buildings. These buildings certainly had far fewer water-trapping features and penetrations.

If the design is based on a no-overhang, California look, the probability of failure will be greatly elevated unless further steps are taken to add deflection at the surface of the wall. This will still be possible if the target market is high-end and can accommodate a drained cavity wall such as strapped cedar siding or brick veneer. However if it is low-end, as has been the case for most construction in the past 12 years, then a lower-cost cladding such as conventional stucco or vinyl will be chosen. The reliance is then fully on the cladding and envelope system to deflect the full load of rainfall. For conventional stucco, drainage and drying will be minimal. The overall capacity will therefore be low (Figure 4). This is the type of construction which has suffered from moisture leakage and deterioration within 5 years of completion. Most of these buildings have had the cladding removed, the decayed framing replaced with treated wood (increasing decay resistance), a vented cavity constructed (increasing drainage and drying) and flashing added or remodeled (increasing deflection). The cladding may be replaced with stucco on a more rigid paper-backed lath, or with vinyl siding. This system is intended to provide sufficient capacity to deliver the desired service life for the building (Figure 4).

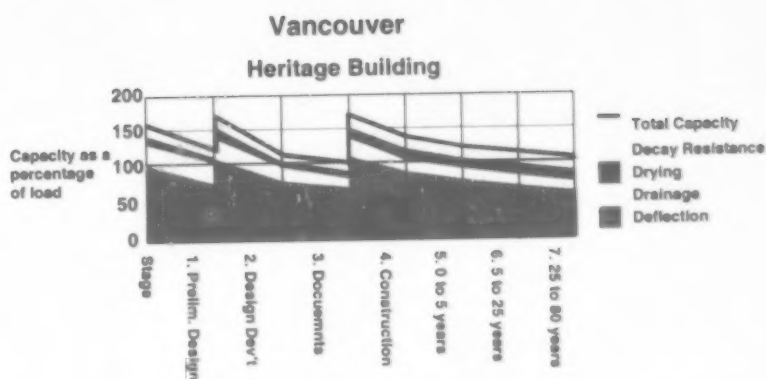


Fig. 3: Moisture load/capacity balance

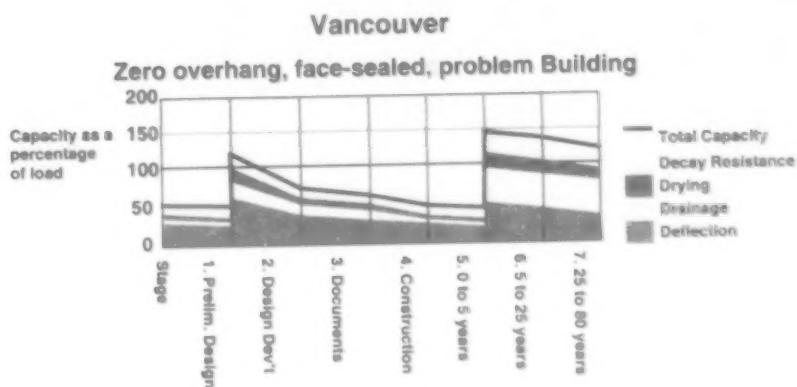


Fig. 4: Moisture load/capacity balance

In areas such as Arizona, where the rainfall is much lower, the conventional stucco building may well have provided the desired service life without such major problems and consequent repairs (Figure 5).

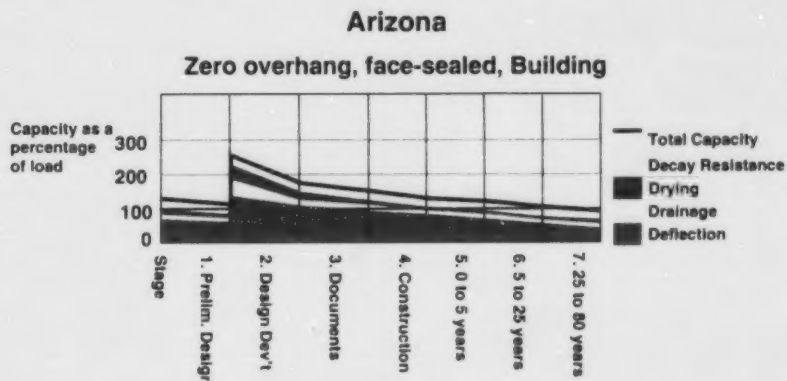


Fig. 5: Moisture load/capacity balance

5.2 Stage 2: Design development

If the California-look development is in a high end market and the design team recognizes the risks of zero deflection then they could choose to introduce a high performance cladding system, with high capacity drainage and high efficiency drying.

To illustrate how this might affect the probability of failure these factors are entered as excess moisture management capacity (Figure 6). At this stage the building has not reached the stage of working drawings and the extent and quality of the detailing is not known. Theoretically the building's load/capacity balance is greater than 100 and the likelihood of failure has been reduced to zero. Unfortunately the process does not stop here with a perfect design.

5.3 Stage 3: Construction documents

Continuing with the example of the California-look, zero overhang high-end rain-screen stucco, the design moves through the process into working drawings. At this point the design could be fully detailed and specified with all the right materials and arrangements shown on the contract documents. If this is the case the load/capacity balance does not change. If on the other hand the design is minimally documented and the specifications are not well coordinated leading to incompatible or incorrect materials choices, the design is compromised and the capacity is reduced. This is shown by a drop in capacity within the column for stage 3 in Figure 6.

The wall has lost some of the critical design intent in the drainage system and is

now back into the high probability of failure category. This may seem to be pessimistic however experience in the field survey (Morrison Hershfield 1996) has shown that failure to carry through the original design intent was a common cause of failure. Incomplete, incorrect or no design details were often accompanied by poor construction.

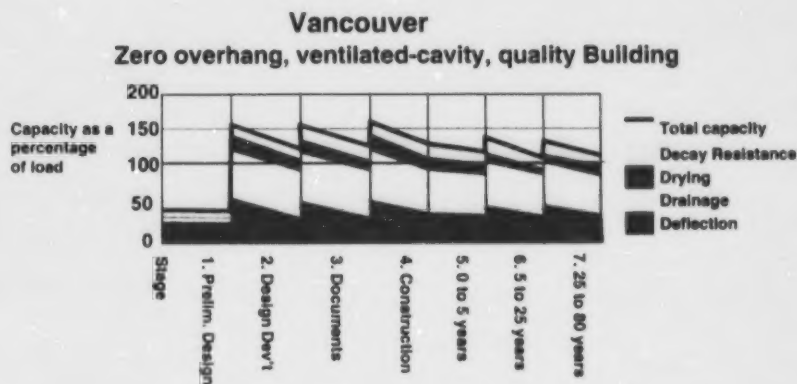


Fig. 6: Moisture load/capacity balance

5.4 Stage 4: Construction

During the construction stage the capacity of each moisture management tool may be enhanced by improvements in design introduced by a knowledgeable site superintendent or craftsman. This stage also has the potential for the greatest degree of deviation from the original design intent. The multiple nature of the trades working on the envelope and the impetus to control and cut costs during construction means that the reliability of the moisture management system is always degraded during the construction phase. This is shown as a drop in capacity for all four moisture management mechanisms in Figure 6. Obvious leaks and discontinuities in the moisture management system will often be corrected within the first 12 months of operation however the hidden moisture management system in most concealed-barrier and drained-cavity walls means that 98% of the system cannot be easily observed or repaired. In the Vancouver case low volumes of water entering the wall over a long period of time led to the demise of numerous wall systems within three to ten years. If the envelope has insufficient capacity to deal with the load, the building will likely fail within the first five years.

5.5 Stage 5: Initial occupation to five years

During the first five years, shrinkage and settlement may cause disruption of the moisture management systems. Aging and weathering of materials may lead to further reductions in capacity, for example, differential thermal properties of adjacent materials may cause cracks to open up. Deterioration of caulking may also reduce the capacity for deflection at joints between materials, indicated as a further drop in deflection capacity in Figure 6.

5.6 Stage 6: Five years to 25-year mortgage period

Planned maintenance activities at the five-year mark may restore some of the lost capacity in the deflection systems (Figure 6). It is highly unlikely that drainage, drying or decay resistance of framing members can be improved without major structural modifications. However, small-dimension exterior wood components may be remedially treated with brush-applied liquids and larger members may have soluble preservative rods inserted into drilled holes.

Aging and weathering over the 5 to 25 year period will cause further deterioration in moisture management systems, but major repairs should not be required (Figure 6).

5.7 Stage 7: Twenty-five years to life expectancy of 80 years

Between 25 and 80 years, it may be necessary to replace the cladding on a building. At that time, it may be possible to introduce additional drainage and drying capacity, if required. However, with attention to moisture management at an earlier stage this should not be necessary (Figure 6). There is also limited opportunity to provide increased decay resistance, but application of surface-applied preservatives to framing will not provide the same level of decay resistance as an initial pressure treatment.

6 Towards a mathematical model for the load/capacity balance

If the load, in terms of annual rainfall, is assumed to remain constant, the probability of success of the building envelope is a function of the excess of capacity over load. The total capacity is the sum of the capacities provided by the four moisture-management strategies: deflection, drainage, drying and decay resistance. However, since the intended capacity is typically reduced by imperfect design and construction, it is necessary to include a capacity reduction term related to the probability of perfection and the impact of imperfection. Each stage when an attempt is made to add additional capacity with each moisture management strategy will have a value for the capacity reduction term and the effective capacity will be the intended capacity multiplied by the capacity reduction term. A mathematical model based on these principles is under consideration. While such a model may well be too cumbersome for general application, it does serve to illustrate the numerous occasions where attention to detail, or the lack of it, can positively or negatively impact the success of a building envelope.

If such a model can be fully developed, the performance of the building envelope system can be predicted for the life of the building. In the design phase this becomes a heads-up warning for the designer, the owner and the builder. For the builder in the construction phase it becomes a quality tracking system for performance management of the sub-trades. In the property management phase it becomes a planning tool for the setting of maintenance schedules and the development of contingency reserves.

The objective of the BERC program is to support the development and continuous improvement of the Best Practice Guides and Quality Assurance Protocols through research into the factors which affect the moisture management strategies defined in the four D's.

7 Conclusions

Design, construction and operation of wood buildings for durability succeed through consideration of the 4 Ds of moisture management, relative to the probability of perfection and the effect of time on long-term performance of moisture management systems.

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70 PREDICTION OF WOOD DECAY IN THE EXTERIOR WALL OF WOODEN HOUSES BY THE DETERIORATION OF NAILS

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Abstract

The deterioration of nails was graded using a five-rank numerical rating. Nails, moisture content and the decay of wood were examined in some mortal-wall wooden houses in central Japan. The wood was mostly in decay when the nail rating was above about four and the moisture content above about 20 %. Nailed wood specimens kept in several conditioning rooms for four years showed that the nail rating was about three in wood with a moisture content of 20 %. A nail rating curve against the service life was obtained the examination of 524 nails in mortal-walls. This curve could be used as a standard service life curve of nails in mortar-walls in central Japan. The curve obtained by connecting the points at rating three in four years and rating four in 34 years seems to be a wood decay warning line in terms of high moisture content. Shear tests for deteriorated nailed joints showed that the load was expressed as a function of the slip and the amount of rust on the nail.

Keywords: bevel siding, decay, mortar-wall, nail, rust, service life, wooden house

1 Introduction

Since the durability of a wooden house depends largely on its joints, nail durability is important especially for the bearing walls (Soshiroda 1950, Hirai and Wakashima 1996). Numerous wooden houses were destroyed during the Hanshin-Awaji (Kobe) Earthquake in 1995 (Murakami et al. 1996). Although most of the houses were old and their construction weak against for big earthquakes, their joint parts had been damaged by wood decay or termites (Hijikuro 1995). Thus the inspection of wooden houses is very important, and easy an inspection method is

strongly in demand. In this study, the deterioration of nails was used as a sensor for the prediction of wood decay. Most iron nails used for wooden houses are standard products and prescribed by the JIS (Japan Industrial Standard), and their quality is stable. The exterior walls of wooden houses in Japan are mainly covered with lath-mortar for protection from fires. In this study, the deterioration of nailed joints was investigated mainly from mortar-walls, and environmental corrosive factors were assessed by the deterioration of nails in the bevel siding walls (Imamura et al. 1987).

2 The relationship between the deterioration of nails and wood decay

2.1 Rating of nail deterioration

A five rank numerical rating scale to grade the deterioration of nails as shown in Table 1 was established. A relationship between the change in diameter d (%) and the rating of the nail (D) was found as shown in eq. 1 and Fig. 1 (Imamura et al. 1983a).

$$d = 0.586D^2 \quad (1 \leq D \leq 5) \quad (1)$$

The length of the CN50 nails was 50 mm. The change in diameter was the average of the changes at the neck, middle and near the point. The relationship between the amount of rust r (% in weight) and the rating of the CN50 nails (D) was obtained as follows.

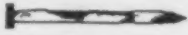
$$r = D^2 \quad (1 \leq D \leq 4.5) \quad (2)$$

2.2 Effective diameter of nails

Morris reported that, in the shearing tests of nails with diameter d , the value of K/d at slip $\delta = 0.6$ (mm) and the load P (N) was stable regardless of d , where K was P/δ (Morris and Gajjar 1973). In fact, however, K/d increased with the d value. From Morris's data, eq. (3) was obtained as follows (Imamura et al. 1983b).

$$K = 155.96 d^{1.57} \quad (3)$$

Table 1: The rating of the deterioration of nails

Rating	Standard	Example
1	Scarcely rusted	
2	Partially rusted, no visible defect	
3	Totally rusted, no defect inside	
4	Partially defect, with original length	
5	Failure	

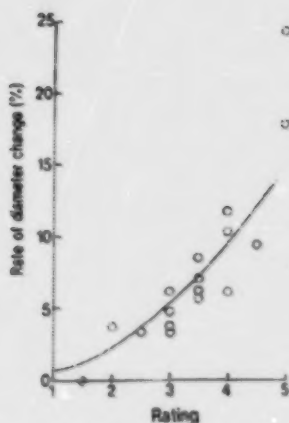


Fig. 1: Relationship between the rate of diameter change and nail rating

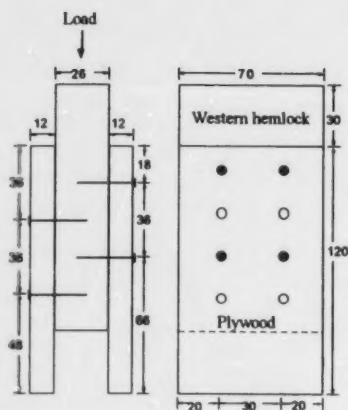


Fig. 2: Shear test specimen of nailed joint

Table 2: Residual percentage of effective diameter of nails

Rating	Kind of nails		
	N32	F50	CN50
1	100	100	100
2	92	103	108
3	82	90	90
4	75	87	73
5	-	74	-

N32 : 32mm in length, 1.83mm in diameter

F50 : 50mm in length, 2.4mm in diameter

CN50 : 50mm in length, 2.87mm in diameter

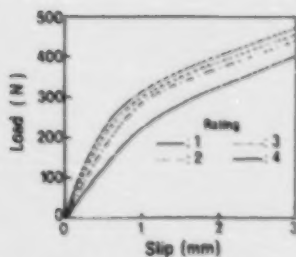


Fig. 3: Load of the deteriorated nail (32mm) of the shear specimen with the sound wood

A shear test of nails (32 mm length) taken from 30-year-old wooden bevel siding was carried out with the specimen shown in Fig. 2. The effective diameter of the nail d was obtained from K at δ (0.6 mm) in the curves in Fig. 3. The diameter obtained from eq.(3) was 1.85 and the one obtained from direct measurement was 1.83. Table 2 shows the residual percentage of the effective diameter.

2.3 Deterioration of wood in contact with rusty nails

Fig. 4 shows the differential thermal analysis curves of the wood powder (Western hemlock: *Tsuga heterophylla*) (Ohkuro and Imamura 1988). The samples were taken from the part adjacent to the nails (A), the part where the rust penetrated (B), and the sound part (C). The main peaks of the curves differ in temperature, and the closer to the rusty nail, the earlier the thermal degradation of the wood occurs. This phenomenon suggests that wood in contact with the rust deteriorated (Marian and Wissing 1960; Farber 1954).

2.4 Deterioration of nails and wood decay

Fig. 5 shows the amount of rust on the nail (CN50) in Western hemlock (160 x 80 x 30 mm) conditioned at several humidity levels at 20 °C for four years. The amount of rust increases rapidly with a relative humidity of 70–80 %, where the equilibrium moisture content of the wood is about 15 %. In Fig. 6, the amount of rust is 9 to 10 % at the moisture content of 20 %. This amount of rust corresponds to the rating of amount three (III) using eq. (2). Since the deterioration of nails and wood is accelerated by water and temperature (Okamoto and Inoue 1977), there seems to be some connection between the degree of deterioration of the nails and that of the wood. Fig. 6 shows the moisture content of the wood and the rating of nails in the mortar-walls of a 34-year-old house. The moisture content was measured by using an electric resistance type moisture meter. The white circles show that the wood near the nails is without decay; and black circles show the wood with decay. Decay or sound was determined by visual inspection. Decay included some damage caused by termites.

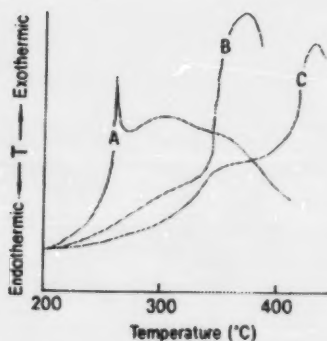


Fig. 4: Curves of differential thermal analysis

- A: Wood in contact with rust nail
- B: Wood colored with nail rust
- C: Wood of a sound part

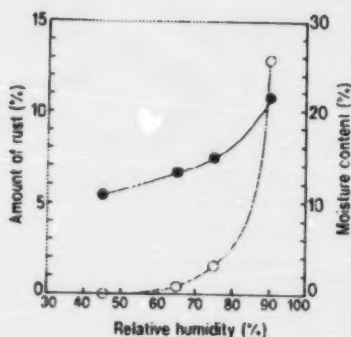


Fig. 5: The equilibrium moisture content

- (●) of wood and the amount of rust
- (○) of nail in wood which were equilibrated in several RH at 20 °C for 4 years.

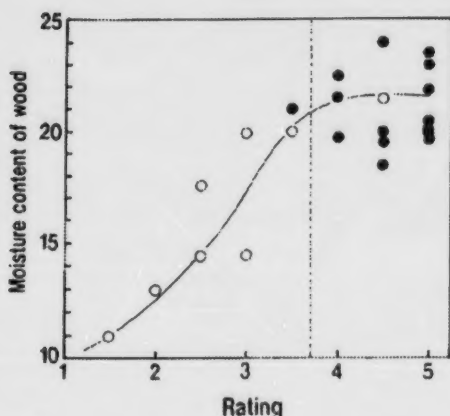


Fig. 6: Relationship between moisture content of wood and the nail rating

(○): Wood without decay

(●): Wood with decay

By using a statistical method, the discriminant function Z_{WD} is expressed as eq.(4) where W (%) is the moisture content of the wood and D is the rating of the nail.

$$Z_{WD} = 0.03 W - 3.76 D + 13.63 \quad (4)$$

According to the discriminant function, the wood is sound when Z_{WD} is positive, and in decay when negative. In Fig. 6, the right side of the dotted line which expresses Z_{WD} is a decay area. The possibility of misdiscrimination is 10.9 %. It is also 10.9%, when the discrimination depends upon the nail rating only. In this case, the discrimination function Z_D becomes zero, at a nail rating 3.75. The discrimination function by the moisture content Z_W becomes zero at the moisture content 19% with the possibility of misdiscrimination of 23.6 %. This moisture content value is close to 20 % which is said to be the critical value in wood decay.

2.5 Shearing properties of the deteriorated joint

The double shear specimens were made by using CN50 nails, plywood (70 x 120 x 12 mm) and Western hemlock solid wood specimens (70 x 120 x 26 mm, average specific gravity: 0.53). The conditions for the deterioration were that (a) the specimens were kept in a sink for one year where hot water (50 °C) flowed beneath the specimens, (b) the specimens were kept in a transparent plastic desiccator for one year which was placed near a sunny window side with water beneath the specimens. The moisture content at the test was 80 – 102 % for (a) and 21 – 26 % for (b). After the shear test, nails were treated with ferric ammonium citrate for removing rust, and the amount of rust was obtained as weight loss.

Fig. 7 shows the load of the deteriorated double shear specimens (solid line) and the specimens made of the sound plywood, Western hemlock and the deteriorated nails (dotted line). Each curve is the average of three specimens. The curves show that the load of the deteriorated specimens was higher than that of the specimens with sound wood in the lower slip region, and the deteriorated wood decreased the load in the higher slip region. The iron rust made the connection tight between the nails and wood mainly effective in the lower slip region, and the deteriorated wood decreases the load in the higher slip region. Fig. 8 shows the relationship between the load and the rating at the slip of 0.9 mm. The load of the deteriorated specimen and that of the specimen with sound wood agree with each other at the slip of 0.9 mm where the effect of the rust and the wood on the load becomes a minimum value. The possibility of estimation of the load at the slip of 0.9 mm from the rating of the nails seems likely to be high regardless of the difference in the specific gravity of the wood.

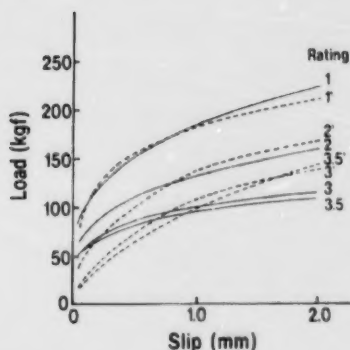


Fig. 7: Load of deteriorated samples (1-3.5) and of sample (1'-3.5') made with the deteriorated nails and the sound wood

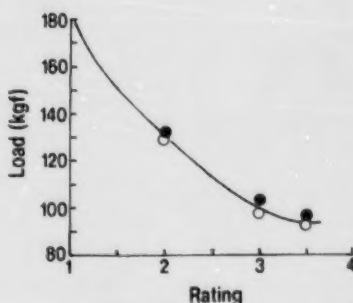


Fig. 8: Relationship between the load at 0.9mm slip and the rating (CN50, double shear)

- : Deteriorated samples
- : Samples made with deteriorated nails and sound wood

3 Effect of the design and details of the wall

One of the most influential factors for the durability of the mortar-wall is a crack in the mortar. Rainwater penetrates through cracks into the wall and deteriorates the nails and wood. Fig. 9 shows the mortar-wall of a house with a crack. The rainwater penetrated through the crack and increased the average nail rating seen on the right side of the floor in Fig. 10. Fig. 11 shows the rating of the nails and the moisture content of the area between the top of the eaves of the first floor and the bottom of the window sash on the second floor in the eastern wall. Penetration of rainwater from the window sash increases the rating and the moisture content.

The rainwater permeates the mortar when it is saturated with water. Traces of permeation were found inside the mortar-wall of the northern aspect of the Northwest corner NW (N), but not in the western aspect of the corner NW (W) of the house inspected. The moisture content of the mortar at the NW (N) was higher than NW (W) as shown in Table 3.

The influence of the interior corner and the rainwater pipe on the nail deterioration was investigated in the bevel siding wall of a schoolhouse. The average ratings on five to six nails at a height within one meter from the top of the foundation were 4.38, 3.94 and 3.93 for the service years of 21, 25, and 28 years, respectively. This indicates that the factor of environmental circumstances is more influential than the service years.

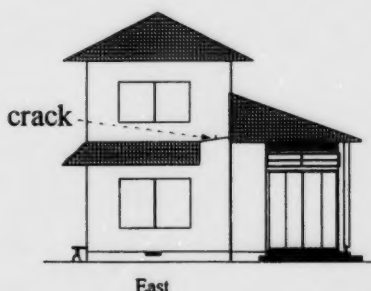


Fig 9: Elevation (Nagoya City)

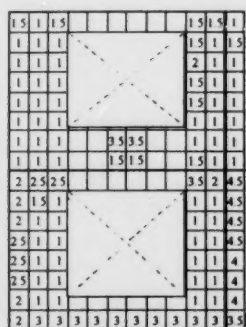


Fig 10: Distribution of the nail rating in the eastern wall

Table 3: Moisture content of mortar

Aspect	Moisture content (%)
NW (N)	3.88
NW (W)	1.84
NE (E)	0.08
SE (S)	0.57
Balcony	4.05

NW(N) : Northern aspect of the northwest corner.

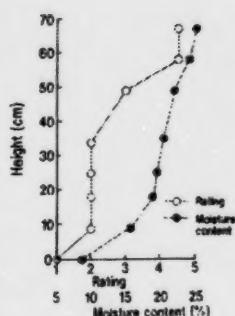


Table 4: Numbers of nails for each rating
(the exterior wall covered with lath-mortar)

Ratings	Years of service							
	3	11	13	13	34	34	34	
1	169	0	0	0	0	0	0	0
1.5	19	0	1	0	0	1	0	0
2	21	0	4	2	1	0	0	0
2.5	11	7	1	3	0	1	0	0
3	55	18	8	7	14	5	17	
3.5	3	20	2	7	4	7	8	
4	6	12	2	5	7	2	15	
4.5	0	0	2	5	4	1	16	
5	0	0	0	0	9	3	19	
Sum	284	57	20	29	39	20	75	
Ave	1.64	3.32	3.00	3.43	3.82	3.55	4.08	
St. dev	0.892	0.473	0.822	0.728	0.843	0.850	0.744	

Fig. 11: The nail rating and the moisture content of plywood under the window

4 Prediction of wood decay from nails

The change in the rating of nails as service life elapses is fundamentally important for the evaluation of a house. Table 4 shows the number of nails for the rating levels in the mortar-walls and Fig. 12 shows the curve obtained from Table 4. The relationship between the rating D and the years of service (t) is expressed in eq. (5).

$$D = 2.2 \log t + 0.6 \quad (5)$$

The black circles in Fig. 12 are the average rating of nails with limited numbers and were not used in obtaining the equation.

The following findings are important to make an attempt to predict wood decay from nails.

- (1) From the inspection of the mortar-wall of the 34-year-old house, the wood decay was found to be prevailing at a nail rating of above about four and moisture content of above about 20%.
- (2) The test showed that the nail rating was about three in the wood kept in 20% moisture content for four years.

The following assumptions are needed:

The moisture content of the wood in the inspected mortar-wall is stable. Namely the moisture content of 20% is significant, and brings the nail to rating four after 34 years.

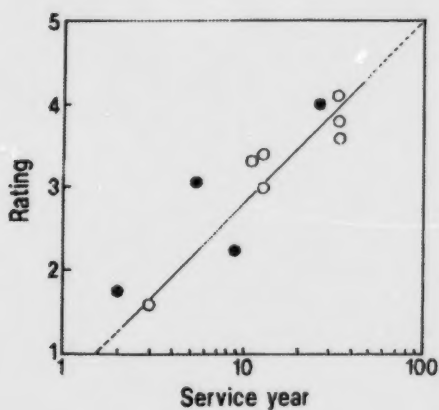


Fig. 12: Relationship between the nail rating in the exterior wall covered with lath-mortar and service life

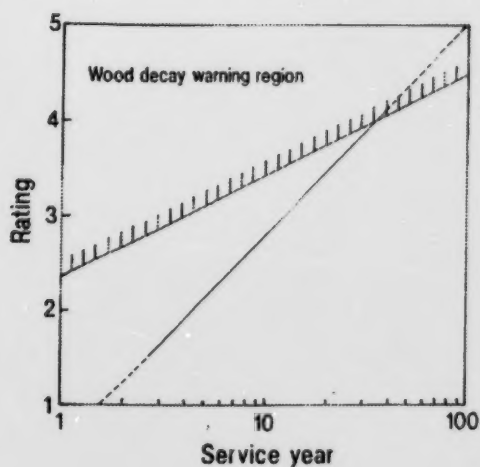


Fig. 13: Relationship between the nail rating and decay of wood in the exterior wall covered with lath-mortar

Based on the above findings and assumptions, a line from rating point three at four years (III at 4 y) to rating point four at 34 years (IV at 34 y) is conceivably a decay warning line of wood in mortar-wall. Fig. 13 shows the deterioration curve of the nail and the decay warning region of the wood. In the inspection of the mortar-wall, if the rating of the nail is in the wood decay warning region, the moisture content is above 20 % and the wood seems to be undergoing decay even if it is sound upon the inspection.

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71 MODELS FOR TIMBER DECAY AND TERMITE ATTACK

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Abstract

This paper describes progress in the development of prediction models for the attack of timber construction by decay fungi and by termites. It was found that attack by decay could be modeled with reasonable confidence by computing the moisture and temperature history and then using field tests to calibrate the behavior models. However for attack by termites, little quantitative information is available and so it was found necessary to calibrate the model by expert opinion.

Keywords: timber, wood, durability, decay, termite

1 Introduction

Computational design procedures for the structural performance of timber buildings are well developed and codified. By contrast, design for durability is very much an art; it is 'good practice' based on experience. However, such a procedure does not cope well with innovation and does not lead to a rational procedure for optimizing building design.

In order to develop an engineering approach to design for durability, it is first necessary to develop computational models for predicting performance. These models, as with all engineering models, are associated with quantified uncertainties, and lead to predictions of performance in probabilistic terms (Leicester *et al.* 1998).

2 Attack by decay fungi

2.1 Models for decay

Probably the two most useful collections of information for engineering models are to be found in the book by Zabel and Morrell (1992) and the report by Viitanen and Ritschkoff (1991).

One of the first decisions to be made is the choice of classification to be used for grouping the numerous decay fungi. For engineering purposes, it would appear useful to group decays according to their mode of attack, i.e., to group decays as brown rots, white rots and soft rots.

The major parameters that determine the rate of attack are as follows

- wood species
- preservative treatment (if any)
- moisture content
- temperature
- available oxygen

Secondary parameters that have a minor influence on the rate of decay include pH and available nitrogen.

With respect to moisture, it is important to appreciate that fungi will not grow well unless there is some free water available, i.e. the moisture content is near or above the fiber saturation point. For this reason it is important to be able to predict the movement of free water in timber. Probably the simplest method to do this is to use the Brook-Corey model of capillary flow (van Genuchten 1980). For this case, it is assumed that the relationship between free moisture content θ , capillary suction head h and hydraulic conductivity k are related by

$$\left(\frac{h}{h_b} \right) = \left(\frac{\theta_{sat}}{\theta} \right)^a \quad (1)$$

for $h > h_b$ and

$$\left(\frac{k}{k_{sat}} \right) = \left(\frac{h}{h_b} \right)^b \quad (2)$$

where θ_{sat} and k_{sat} denote values at saturation, h_b is a 'bubbling pressure' and a and b are related constants.

Examples of the fit of equations (1) and (2) to published data are shown in Figs 1 and 2, respectively. The advantage of using these simple equations is that they require the measurement of only a limited set of parameters. These may be obtained through simple tests, such as sorption tests. In addition, the solution of practical cases may be undertaken through the use of techniques that have been well developed for soil mechanics, such as through the use of the Kirchoff transform (Ross and Bristow 1990).

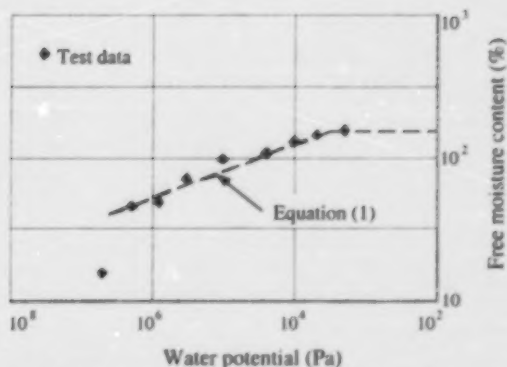


Fig. 1: Relationship between moisture content and free water suction for tangential desorption of Aspen sapwood (after Cloutier and Fortin 1991)

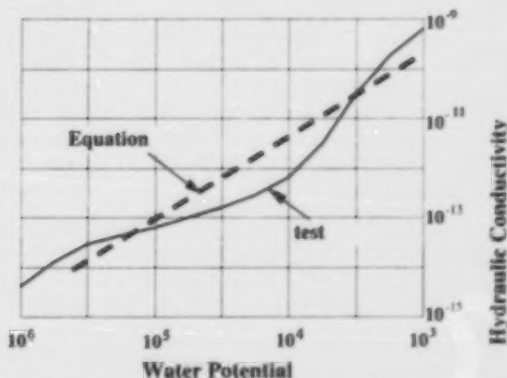


Fig. 2: Relationship between free water suction and tangential hydraulic conductivity for Aspen sapwood (after Cloutier and Fortin 1993)

The following discusses some of the aspects of predicting loss of effective section through decay. Once this prediction is made, it is possible to compute the residual strength and hence the safety of a structural element.

2.2 Exposed timber in ground contact

For timber in ground contact, such as for example the pole shown in Fig. 3, it is necessary to model the movement of moisture through the soil and into and out of the timber. Rapid and effective computer programs for the movement of moisture into soils and through crops already exist (APSRU 1997). These programs take into account climate, soil parameters and evapotranspiration effects; they also include computation modules for predicting the nitrogen content and pH of the soil.

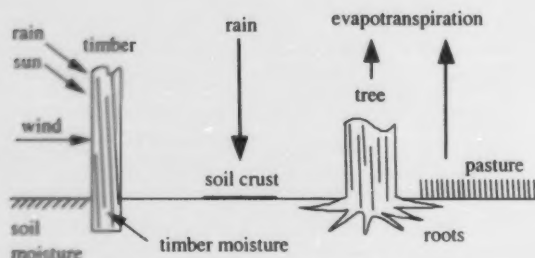


Fig. 3: Example of exposed timber in ground contact

For the development of effective models of decay, it is essential to have data from field studies for use in calibrating the model. Two sets of data currently available are those from graveyard stake tests and those from the study of power poles. Australian field sites for graveyard stake tests and above ground L-joint tests are shown in Fig. 4. Data from graveyard stake tests on untreated timber at one particular location are shown in Fig. 5. In this and subsequent figures, the depth of decay refers to the depth as measured by a pen knife or screw driver probe.

Typically, the attack on untreated wood starts as soon as there is ground contact. In general, low durability softwoods are attacked by brown rot and low durability hardwoods by white rot; high durability hardwoods are attacked by soft rot.

An example of data from graveyard stake tests on CCA treated timber for a site near Sydney is given in Fig. 6. The attack appears to be developed by about 6 years from the time of first ground contact. Since the delay is roughly the same for all retentions of CCA, it is possible that the delay is due to the time for the soft rot attack to establish itself, rather than the leaching of the CCA.

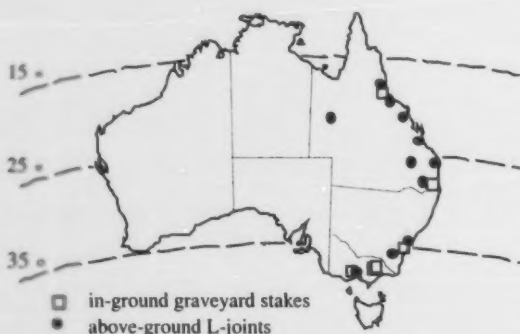


Fig. 4: Site locations for Australian exposure tests

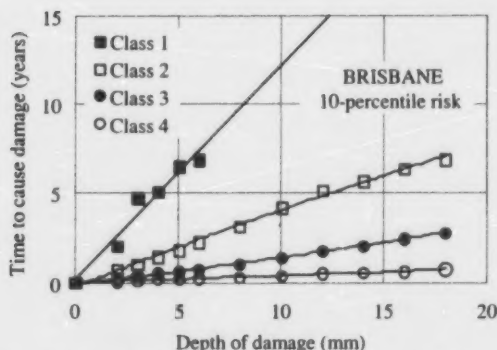


Fig. 5: Example of biological attack on untreated graveyard stakes (after Thornton, J. and Johnson, G., private communication)

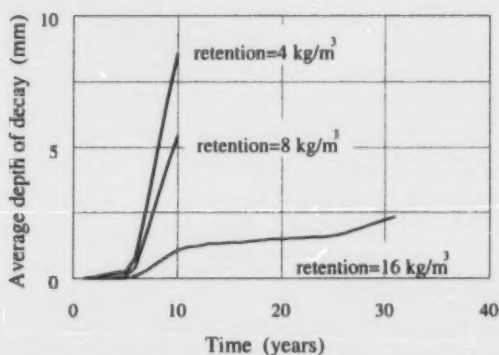


Fig. 6: Decay of CCA-treated Radiata Pine graveyard stakes at Pennant Hills, NSW (after Thornton, J. and Johnson, G., private communication)

There is however, no doubt that leaching of CCA in-service does occur (Lebow 1993). Furthermore, it does appear that there may be a relationship between residual CCA and decay as illustrated by the example shown in Fig. 7. The relationship, if any, between durability and loss of CCA would be a useful feature to incorporate in a decay attack model.

Of direct importance to the engineer is the variability of the data. An example of this is shown in Fig. 8 for a site in northern Queensland. For this case, the measured penetration of decay after 30 years has a coefficient of variation of about 100% and as a result, an exponential curve is seen to provide a reasonable fit to the data.

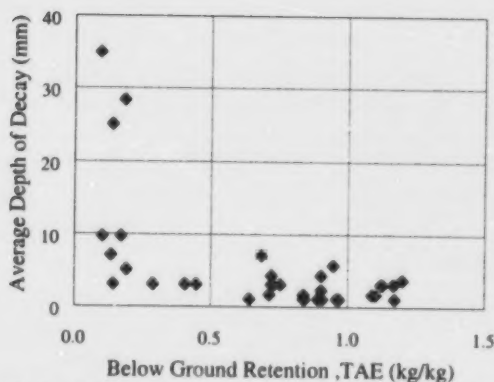


Fig. 7: Observed relationship between residual CCA and depth of decay after 20 years in Spotted Gum poles located at Wedding Bells test site in NSW; retention refers to residual weight of active copper chrome and arsenic elements in (kg/kg) (after Gardner, D., private communication)

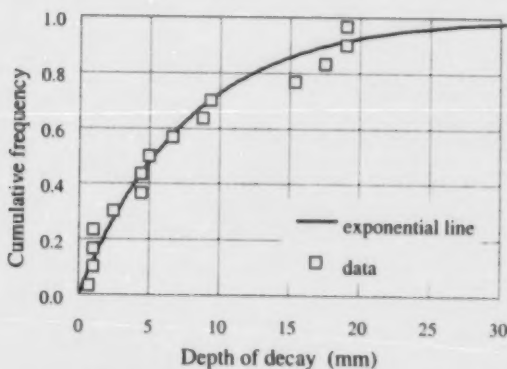


Fig. 8: Decay of CCA-treated Radiata Pine graveyard stakes at Innisfail, Queensland, after 30 years; initial retention of CCA was 16 kg/m^3 (after Thornton, J. and Johnson, G., private communication)

2.3 Above-ground, exposed timber

For the case of above-ground exposed timber, it is necessary to compute the time-of-wetness for each timber element. This can be done from rainfall and wind data. An example of this is shown in Fig. 9. To compute the time-of-wetness under a roof overhang, it is necessary to take into account the wind direction, wind speed and terminal velocity of the raindrops.

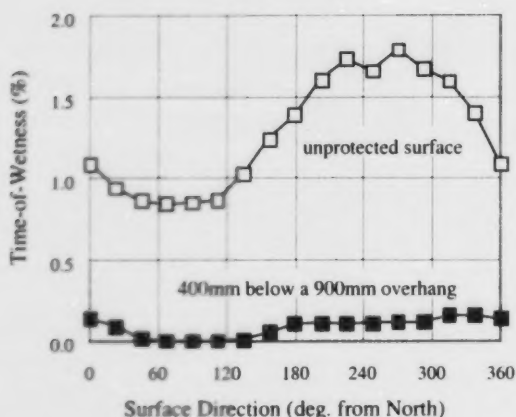


Fig. 9: Time-of-wetness for a wall surface in Melbourne in 1972

Several other factors must also be taken into account. The effect of surface coating and/or surface treatment is important in reducing the sorption of surface moisture (Derbyshire and Miller 1997). The effects of climate induced cracks affect the penetration and retention of surface moisture. Finally, the surface temperature of the wood is important since fungi will be killed when exposed to temperatures above 80°C for half an hour, conditions frequently achieved in many parts of Australia. To compute the surface temperature with reasonable accuracy it is necessary to take into account the degree of cloud cover, and also the radiation characteristics of the surface paint or finish.

A major obstacle to modeling above-ground decay is that there is little published biological data that can be used directly for this purpose. The decay fungi involved are different to those associated with attack in in-ground elements, and hence the data obtained from graveyard stake tests cannot be used to estimate the effects of attack on above-ground timber. Probably the most useful data available for calibration purposes is that obtained from L-joint tests (Cause 1993).

2.4 Protected timber

For the case of protected timber in buildings, there is now an extensive body of technology for predicting moisture and temperature effects (e.g. ASHRAE Handbook 1993; Tenwolde 1983). The risks of decay for protected timber are generally small unless there is frequent condensation to provide access to free water. Unfortunately, there is little quantitative information on the characteristics of decay fungi that attack internal timber.

3 Attack by Termites

For the case of termites, the model chosen is that T , the time to destruction of a building element, is given by

$$T = T_1 + T_2 + T_3 + T_4 + T_5 \quad (3)$$

where $T, T_1, T_2, \dots$ are all random variables.

The definition of each event time T_i , and the parameters assumed to affect it, are shown in Table 1. Obviously each event time is the result of a balance between biological and environmental factors. However, there is little quantitative information in the literature that can be used for calibrating this model. Accordingly, this calibration is being undertaken through surveys of the opinions of experts on termites.

Using such surveys, T_i may be computed for each event as

$$T_i = A_i(X_1 X_2 \dots X_n)_i \quad (4)$$

where A_i is a random variable obtained from the spread or statistics of opinion survey data, $X_1 X_2 \dots X_n$ are parameters that affect the event (the values of which reflect their relative importance or impact on the event, the values are calculated based on survey data) and n is the number of relevant parameters affecting the event (e.g., see Table 1).

Table 1: List of parameters used to model attack by termites

Event Time	Parameters That Affect Event Times
T_1 (Time for establishment of a first nest)	• climate (temperature) • number of potential nesting sites • age of suburb
T_2 (Time to travel to building)	• distance of nest site from building • climate (temperature) • soil type • food sources
T_3 (Time to penetrate anti-termite barriers)	• climate (temperature) • inspection routine • maintenance schedule
T_4 (Time to effect an entry into the building)	• climate (temperature) • nature of ground contact elements • inspection routine
T_5 (Time to destroy a timber member)	• climate (temperature) • environment of timber • type of timber

One of the few sources of quantitative information available is that obtained in an Australia wide survey of termite damage by school children organized by J.R. French. One result from this study is shown in Fig. 10.

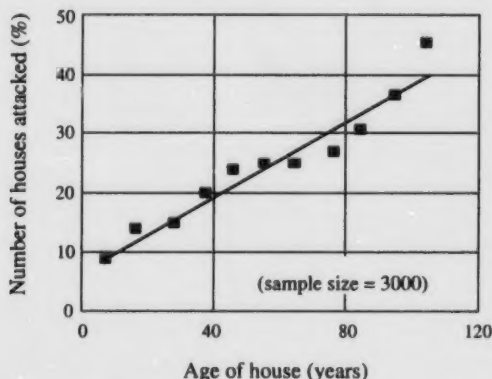


Fig. 10. Effect of the age of a house on the probability of attack by termites (after MacKenzie 1997)

4 Conclusion

Studies to date indicate that there could be enough information to develop quantitative prediction models for the attack of timber by decay fungi, both for in-ground and above-ground cases. However, quantitative information for modeling attack by termites is limited, and to obtain useful accuracy in predictions it may be necessary to undertake extensive field observations directed towards this end. Updated models and analysis results will be presented in the Conference.

5 Acknowledgments

This paper is based on work undertaken as part of a major national project on durability sponsored by the Forestry and Wood Products Research and Development Corporation (FWPRDC) of Australia. The authors are indebted to the numerous participants in the project who have generously provided data for modeling purposes.

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72 DURABILITY ASSESSMENTS OF WOOD-FRAME CONSTRUCTION USING THE CONCEPT OF DAMAGE-FUNCTIONS

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Abstract

Review of the state-of-the-art on fungal damage of wood materials showed that a combination of period of cumulative time when moisture and thermal regimes are above specified minimum is needed for fungal activities to cause biological decay in wood. The long-term performance of a typical wall assembly was evaluated using an existing computer model of heat, moisture, and air transfer in conjunction with a biological damage-function model. The hygrothermal behaviour and the associated biological damage were evaluated using virgin materials properties. The damage results are rather conservative because test results revealed that subsequent wetting and drying change the microstructure features of typical engineered wood products.

Keywords: construction, building, envelope, house, wall system, material, wood, wood-frame, durability, moisture, temperature, time, lifecycle, mold fungi, wood-rot fungi, analysis, fracture, failure, damage, model.

1 Introduction

The problem of durability is not just due to over-stressing of structures. A wide range of physical, biological, chemical, and mechanical processes, as well as improper design and construction practices are responsible for the onset of the damage in building envelopes. Moisture and temperature fluctuations as well as mechanical loads are leading causes of the continuous deterioration of building materials. About 90% of damage in houses, with the exception of that caused by mechanical loading, result from temperature and moisture effects (Zabel and Morrell 1992). Wood and wood based products (engineered wood) are widely used in most housing and low-rise residential and office buildings in Canada. Wood materials are

very susceptible to degradation due to fungal attack under certain combinations of time, moisture and temperature conditions. Houses may experience moisture-related damage within 3-10 years of service life. The building continues to be serviceable for a longer period but its durability remains an economical issue. Damage prevention of building envelopes is the most cost-effective measure compared to repairing or replacing the whole envelope. Thus, improvements in the analysis, design, and damage predictions of materials and structures can have great economic and safety implications.

This paper reviews the state-of-the-art on fungal damage of wood materials. It discusses the critical time needed for given moisture and thermal regimes for fungal activities to cause damage in wood. The paper, as an example, assess the long-term performance of wood-frame constructions in Ottawa, Canada using fungal damage models (Hukka and Viitanen 1998).

2 Damage types in wood materials

Engineered wood materials are susceptible to degradation under certain hygrothermal and/or mechanical loading conditions. For example, oriented strand board (OSB) and plywood exhibit dimension changes when they get wet.

The onset of biological damage is considered differently by various researchers. For instances, Zabel and Morrell (1992) stated that the process of biological damage commences when moisture content reaches more than 35% by mass in wood materials. Baker (1969) reported that moisture content of less than 20% might not cause structural wood decay. However, Zabel and Morrell (1992) reported that some experiments showed that moisture contents below 20% could become suitable for bacterial and fungal growth that ultimately destroys wood materials. This result is debatable because no other work has ever been able to repeat mold growth for this particular condition. Measurements in typical housing indicate that moisture content of wood varies between 5% in the winter and 15% in summer months. Improperly designed or built wood envelopes, however, may accumulate and retain higher moisture content, leading to rotting. Baker (1969), and similar work conducted by Zwick (1985), indicated that moisture condensation is not a problem when properly dried wood is used for the construction. Although the wood structure may be capable of resisting loads, the home becomes uninhabitable because of the poor health environment that results from fungus spores.

Nofal (1998) showed that engineered wood materials suffer biological and structural damage when subjected to hygrothermal loading. Also, Nofal (1998) summarized the damage-accumulation approach to model the time-dependent strength properties of wood structures. Structural damage will not be further discussed here because it is outside the scope of the current paper. This paper will focus only on the biological decay aspects and their influence on the long-term performance of engineered wood materials.

3 Biological damage in wood materials

A wide range of organisms including fungi, termites, carpenter ants, wood-boring beetles and marine borers can live in and consume wood. Water is the essential element for the growth of most of those organisms in wood. Termites are a serious threat to untreated wood in tropical locations (Morris 1998, Zabel and Morrell 1992). However, termites cause also serious problems in temperate climate such as in Canada especially in some parts of southern Ontario, particularly downtown Toronto, and in some of the drier areas in British Columbia (Morris 1998). Grubs are the most larvae of wood-boring beetles that feed on the wood. Marine invertebrates chew tunnels in wood stored or used in salt waters. Carpenter ants do not eat wood but they excavate it to live in. Fungi are the most important organisms that destroy wood under relatively mild climate condition.

Most of the reported damage in European and North American houses and low-rise buildings are mainly due to fungal attack on the wood materials. Therefore, this paper summarizes only the-state-of-the-art on fungal damage in wood materials.

4 Studies of fungal damage

Fungi are those organisms that feed on organic matter and grow by extension and branching of tubular cells (*hyphae*). Fungi reproduce through microscopic spores that are smaller than the pollen grains of plants. Fungi species differ from one place and material to another depending on the microclimate. In general, fungi that live in wood can be divided into four groups: mold, stain, soft-rot, and wood-rotting basidiomycetes fungi.

4.1 Mold-fungi damage

The presence of mold fungi in homes is an early indication of increased humidity and moisture levels in these buildings. Mold fungi are undemanding organisms and are able to grow on any building material, including wood. Mold fungi growth on non-organic materials will not start unless there is some nutrient existing on their surface (e.g. dirt). Growth rate of mold fungi depends on the surface quality (kiln dried or fresh sawn) not on the wood species (Viitanen 1997 a). Wood surface quality reflects the nutrient content of the surface. White (1996) and Andersson *et al.* (1997) also found that mold fungi could reproduce and evolve on any surface, depending on the amount of free sugar regardless of material type. Mold fungi will rapidly develop at a relative humidity of 75% or higher. These fungi types go dormant under dry conditions but they will be replaced by wood rotting fungi if the wood material remains wet. Mold fungi cause damage in terms of discoloration, odor and health but do not significantly affect the strength of wood (White 1995, 1996, Andersson *et al.* 1997).

Viitanen and Ritschkoff (1991) divided mold and stain fungi into wood colonizing fungi and airborne mold fungi. They collected from European homes some typical wood colonizing mold fungi that include *Alternaria alternata*, *Aspergillus* -

species, *Aureobasidium pullulans*, *Cladosporium cladosporioides*, *Chaetomium globosum*, *Paecilomyces variotii*, *Penicillium* -species and *Tirchoderma viride*. Schmidt *et al.* (1983) indicated that these mold fungi species could be found in North American homes. The intensity of the fungal damage may be different for the two continents because of differences in weather and occupants living habits and wood species. These differences need to be studied.

4.2 Wood-rot fungi damage

Wood-rotting basidiomycetes cause most serious damage in buildings. These fungi types can be classified into three main categories: (1) soft-rot, (2) white-rot, and (3) brown-rot fungi. Oak and maple species are very susceptible to soft-rot fungi. White-rot fungi destroy hardwoods such as aspen and maple. Brown-rot fungi prefer softwoods such as pines, spruces, hemlocks, and firs. All these fungi have considerable ability with varying degrees on reducing the wood material strength. Wilcox (1978) showed that white- or brown- rot fungi affect several strength and mechanical properties of wood before significant weight loss is detected. The slow reduction in material strength is the result of the break up of lignocellulose, lignin and cellulose depending on the fungi type.

Duncan and Lombard (1965) identified *Antrodia serialis*, *Antrodia vaillantii*, *Coniophora puteana*, *Gloeophyllum trabeum*, *Meruliporia incrassata*, *Postia placenta*, *Sepula lacrymans*, and *Tapinella panuoides*, as the fungi causing decay in building in the United States of America. In the Nordic countries, *Coniophora puteana* fungi are the most lethal fungi species attacking homes (Viitanen 1997 b). Morris (1998) indicated that there is no extensive studies have been done to determine which fungi species are the most important in Canada. Morris (1998) suggested that those fungi species identified from USA buildings might be common to Canadian environments.

4.3 Optimal conditions for fungal growth

Some of these fungi can survive and grow at low moisture concentrations causing decay and weakening or discoloration of the building envelope materials (Zabel and Morrell 1992). The process of fungal attack in building envelope requires five critical requirements: (1) wood type, (2) free or unbound water, (3) favorable temperature, (4) oxygen and (5) available digestible carbon compound. Wood-inhabiting fungi require optimal levels of those essential elements for viable growth.

4.3.1 Optimum moisture conditions

The exact optimal moisture levels for the growth of fungi are not well known (Zabel and Morrell 1992). Scheffer (1973) reported that in laboratory decay tests, the optimal moisture contents lie between 35% and 80%.

Most wood-inhabiting fungi cannot grow effectively in water-saturated wood because they are obligate aerobic organisms that require moderate amounts of oxygen for respiration. The upper limits of moisture contents that may cause decay vary between 100% and 250% depending on the void volume of wood, which varies inversely with wood specific gravity. Higher moisture contents will slow fungal

growth in high-density wood. Optimal moisture levels for fungal growth in actual building envelopes are not exactly known and need to be identified.

4.3.2 Optimum oxygen levels

Another critical ingredient for fungal growth is the amount of free oxygen available for their metabolism. The process of aerobic respiration uses atmospheric oxygen as a reactant. Many fungi have optimal growth rate at oxygen levels between 19% to 20% of ambient air. Fungi are, however, very tolerant of low oxygen levels and their growth decreases when oxygen concentration drops below 1% to 2%.

Fungi growth rate depends also on the combination of oxygen and carbon dioxide. Build-up of carbon dioxide limits the amount of oxygen and consequently stops fungal growth but fungi are not necessarily killed.

4.3.3 Optimum temperature ranges

Fungal growth also depends on temperature levels. Temperature directly affects the many integrated metabolic activities of fungi such as digestion, assimilation, respiration, relocation, and synthesis. The metabolic reaction rate increases with increase in temperature until the heat denatures enzymes required for growth. Every fungus possesses three cardinal growth temperatures: a minimum, an optimum and a maximum. These levels indicate, respectively, the beginning of growth, the best growth, and the ceasing of growth.

Ambient air temperature limits for the growths of most fungi are between 0°C and 45°C. The optimum temperatures for fungal growth lie between 20°C and 30°C. Some Fungi such as *Gloeophyllum* species can grow faster at higher temperature ranges of 34°C to 36°C. Growth rate of all wood-inhabiting fungi will stop at temperature higher than 46°C and will be killed at temperature higher than 60°C. Growth rate is often reduced in halves for each ten degrees drop in temperature.

Apart from temperature and moisture content, time is an important factor in fungal wood decay. Fungi are resistant to prolonged exposure to lower temperatures and are killed by short exposure to high temperatures. For example, staining fungi are particularly susceptible to high temperatures and some species are killed by prolonged storage at 35°C. Some wood-rotting fungi exhibit strange resistance to lower temperatures and can resume growth within a few hours after warming.

4.3.4 Other factors affecting fungal growth rate

Fungal growth also depends on other factors, such as the pH value, vitamins, minor metals, and light. In general, optimum fungal growth requires pH ranging from 3 to 6. Mold and wood-stain fungi are highly pH sensitive and their growth ceases or diminishes as pH exceeds 5. Brown-rot fungi, on the other hand, have the lowest optima of pH, approximately 3.

Wood-decay fungi can utilize many types of nitrogen, especially the amino forms found in wood (Huntgate 1940). Light, especially the ultraviolet wavelengths at high intensities, is usually harmful to the growth of wood-decay fungi. Some studies (Duncan 1967), however, suggest that periodic exposure to light may increase decay rate. Current knowledge cannot explain this apparent contradiction and further

research is needed. Other environmental factors may also affect the growth and production of wood-decay fungi. Osmotic concentrations, atmospheric pressure, sound vibrations, gravitational forces and radioactivity are examples of these additional effects. Little is known, however, about their effects on the reproduction of fungi and the wood decay process.

4.4 Fungal damage in housing

Relatively new homes in many regions in North America and Scandinavia show signs of damage induced by fungal activities in their wall systems. Canada Mortgage and Housing Corporation (CMHC) monitored the health of the occupants of 400 moldy houses located in Wallaceburg, Ontario. The study measured the development of mold fungi that is hazardous to the health of occupants (Fugler 1996, White 1996). Their damage assessment was related to the allowable tolerance for carbon dioxide levels in an infant's lung. They did not quantify the growth of these mold fungi for specific construction materials or environmental conditions. They based their finding on the assumption that mold fungi appearance indicates a rise in humidity in the dwellings. Andersson *et al.* (1997) conducted a similar study to measure the toxicity effects of mold fungi growth on the health of toddlers at a child's daycare center in Helsinki, Finland. They investigated the toxic level that emanated from water-damaged gypsum boards attached to wood-frame construction. The results of these two useful studies were based on subjective evaluation of the damage response and so cannot be easily linked to hygrothermal loads.

Other researchers (e.g. Viitanen and Bjurman 1995) attempted to evaluate fungal damage in pine and spruce softwood materials due to growth of known fungi species with respect to moisture control. The main objectives of these studies were to evaluate the critical time required for common mold and wood-decay fungi to latch onto the surface of wood materials subjected to specific environmental conditions. They measured the growth activities of harmful bacteria and fungi to the applied moisture and thermal loads. They developed empirical damage functions of mold and wood-decay fungi in terms of moisture and thermal loads using laboratory experiments. These models will be employed later in this study to assess the long-term performance of wood frame construction in Ottawa, Canada.

5 Measurements of fungal damage

According to Morris (1998), Viitanen (1997 a) and Andersson *et al.* (1997), wood damage will occur provided that mold or wood-decay fungi succeed in latching on the wood to start their colony under favorable conditions. Biological damage in wood is often measured using laboratory experiments because the exact analysis of combined effect of humidity, moisture content, temperature, time and bio-deterioration of material in buildings is difficult. Other studies (e.g. Schmidt *et al.* 1983) focused on measuring the strength reductions in particleboard as a result of fungal damage under constant moisture conditions that favor fungal growth. They found out that particleboard would lose strength significantly when attacked by

wood-rotting fungi.

Moisture and temperature distributions in wall system vary with season and many other factors. Moisture and temperature fluctuate depending on indoor air conditions, exterior weather, and wall system details. Thus, the results of previous studies using constant moisture and thermal loads cannot be used effectively to develop damage functions for actual building envelopes. Viitanen (1997 a, 1997 b) tested pine and spruce sapwood under changing moisture and thermal loads. In these experiments, two separate sets of testing for mold and wood-rotting fungi were conducted. The reason is that mold fungi growth depends on the relative humidity of the environment while wood-decay fungi activities are functions of the moisture content of wood (Viitanen and Bjurman 1995, Viitanen 1997 a, 1997 b). However, in actual life cycle of fungal growth, the wood would be attacked first by mold fungi and then may be replaced by the dangerous wood-rotting fungi.

6 Durability assessments using damage functions models

The TCCC2D two-dimensional heat, air, and moisture model (Ojanen and Kumaran 1992, 1996) was used to analyze the wall system shown in Figure 1. Ojanen and Kumaran (1992, 1996) used this model to determine the effect of exfiltration on the hygrothermal behavior of wall systems. Here, the effect of different indoor relative humidity and varying rates of exfiltration on the behaviour of the wall of wood-frame construction was investigated for Ottawa weather. Figure 1 also shows the tested range of recommended indoor relative humidity (ASHRAE 1995). The results of the TCCC2D computer model were the input for another computer program developed for the purpose of damage assessments in wood-frame constructions. The biological damage module in the new computer damage model was based on the theoretical mold index model proposed by Hukka and Viitanen (1998). The authors for easy implementation in heat air and moisture transport modified the mold index model of Hukka and Viitanen (1998) computer models. A summary of the enhanced mold index model and its damage prediction capabilities will be presented here. A detailed discussion is given elsewhere.

6.2 Enhanced mold damage model

Viitanen (1997 a, 1997 b) developed a mold index model based on the results of decay experiments of pine and spruce sapwoods. The mold index will be interpreted in this paper as the damage index. Viitanen's (1997 a, 1997 b) models are mathematical in nature and were developed to fit his test results. The test samples were subjected to relative humidity in the ranges of 75% to 100% and temperature varying from 5°C to 40°C. Hukka and Viitanen (1998) extended Viitanen's (1997 a) damage model to estimate mold growth under fluctuating moisture and temperature.

The mold index of Hukka and Viitanen (1998) can be calculated by

$$M_{\text{index}} = 1 + 7 \left(\frac{RH_{\text{max}} - RH}{RH_{\text{max}} - 100} \right) - 2 \left(\frac{RH_{\text{max}} - RH}{RH_{\text{max}} - 100} \right)^2 \quad (1)$$

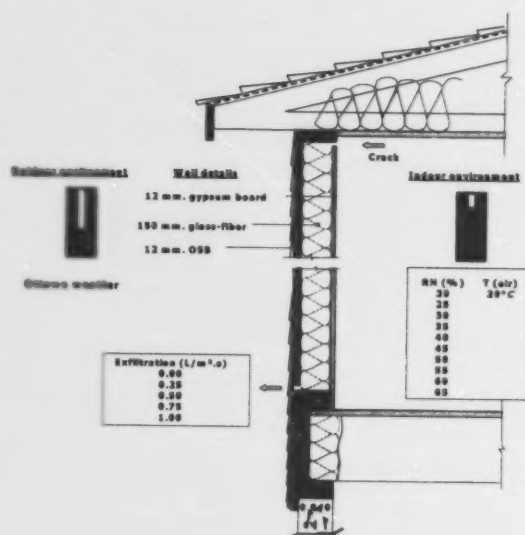


Fig. 1: Typical wall analyzed for the specified boundary conditions

where M_{max} is the mold index that varies from 0 to 6 (Hukka and Viitanen 1998) depending on the appearance of mold on material surface and RH is the test relative humidity. RH_{crit} is the critical relative humidity obtained from experiments as a function of the test temperature, T .

$$RH_{crit} = \begin{cases} -0.00267T^3 + 0.1607T^2 - 3.137T + 100 & T \leq 20^\circ C \\ 80\% & T > 20^\circ C \end{cases} \quad (2)$$

If the ambient relative humidity maintained above the critical value, RH_{crit} , for a period, t_m , fungi will start their reproduction process. Viitanen (1997 a) proposed formula fitted to experimental results to estimate the critical time, t_m , that is needed for initial response of mold or wood-rotting fungi. Hukka and Viitanen (1998) used the initial time, t_m , to calculate the rate of mold growth under favorable conditions using

$$\frac{dM}{dt} = \frac{k_1 k_2}{7t_m} \quad (3)$$

in which k_1 and k_2 are constants depending on the growth rate level and could be calculated as

$$k_1 = \begin{cases} 1 & M < 1 \\ \frac{t_m}{t_v - t_m} & \dot{M} > 1 \end{cases} \quad (4)$$

$$k_2 = \begin{cases} 1 & M < 3 \\ 1 - e^{2.5(M - M_{max})} & M \geq 3 \end{cases} \quad (5)$$

where M is the previous damage index and t_v is the time required for visual growth of mold fungi. t_m and t_v can be calculated according to Viitanen (1997 a) as

$$t_m = e^{-0.68 \ln(T) - 13.9 \ln(RH) + 0.14W - 0.33SQ + 66.02} \quad (6)$$

$$t_v = e^{-0.74 \ln(T) - 12.7 \ln(RH) + 0.06W + 61.50} \quad (7)$$

in which W and SQ are constants that take values either 0 or 1 to reflect the wood species and its surface quality, respectively. When the relative humidity levels are below the critical value, RH_{crit} , (dry condition) Hukka and Viitanen (1998) proposed the following mathematical form for the decline in mold growth rate.

$$\frac{dM}{dt} = \begin{cases} -0.032 & t - t_1 \leq 6 \text{ h} \\ 0.000 & 6 \text{ h} \leq t - t_1 \leq 24 \text{ h} \\ -0.016 & t - t_1 > 24 \text{ h} \end{cases} \quad (8)$$

where $(t - t_1)$ represents the drying period.

Figure 2 shows the damage index variation at the bottom of the wall, shown in Figure 1, due to indoor relative humidity of 50% for different air leakage. The temperature and relative humidity output from the TCCC2D simulations are coupled with the mold damage index model. Figure 2 indicates that mold damage increases in a nonlinear fashion and depends on the rate of air leakage and season. Mold damage occurs in the spring and fall period, as illustrated by Figure 2, which agrees with observations in actual houses.

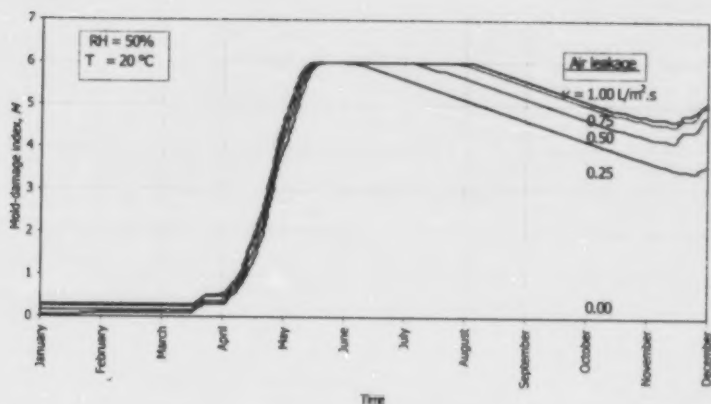


Fig. 2: Damage (mold) index variation with time for different air leakage rate

The simulation results also showed that the airflow rates as well as internal relative humidity affect the extent of mold growth. This is demonstrated in Figures 3 and 4 using the damage index at the end of simulation period (December). Figure 3 shows the variation of the damage index with respect to air leakage for different relative humidities. The figure indicates that damage growth rate is a nonlinear function of air leakage. According to the results shown in Figure 3, damage will not occur for a combination of air leakage rate less than $0.2 \text{ L/m}^2\text{s}$ and indoor relative humidity below 35%. Furthermore, air leakage equal or less than $0.001 \text{ L/m}^2\text{s}$ will not cause growth of mold for all possible indoor relative humidities. Figure 3 shows as expected that higher levels of indoor relative humidity would increase the probability of mold damage.

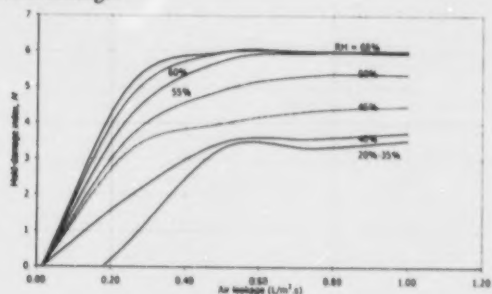


Fig. 3: Effect of air leakage on mold damage index

Figure 4 depicts damage index variation with respect to indoor relative humidity at the end of the simulation period. The figure shows that the damage index appreciably increases when the indoor relative humidity varies in the range 35% to 45%. Damage is much higher for higher levels of indoor relative humidity. The figure shows that damage values are the same for relative humidity in the range of 20% to 35%. Figure 4 shows that for all practical purposes, air leakage in the range of 0.5-1.0 $\text{L/m}^2\text{s}$ have the same effect on mold growth. Finally, Figure 4 shows also that damage values for a combination of indoor relative humidity equal to 20% or lower and airflow rate less or equal to 0.2 $\text{L/m}^2\text{s}$ are insignificant.

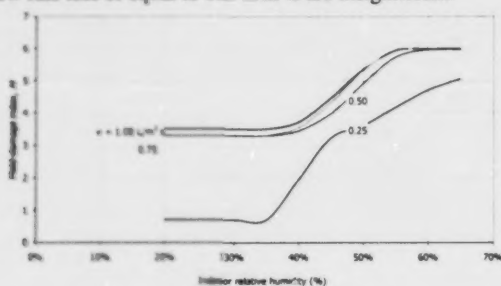


Fig. 4: Effect of relative humidity on mold damage index

It is very important to mention, once again, that the results presented for mold damage index variation are based on pine species experimental results. These results are conservative for OSB materials because the wood species often used in producing OSB panels are less durable than those of pine species.

7 Concluding remarks

In this paper, an approach has been presented to link hygrothermal models with damage functions concept. To illustrate this approach, results from a well-established hygrothermal model (Ojanen and Kumaran 1992, 1996) have been combined with a modified model of mold growth. The original mold model was reported in the literature (Hukka and Viitanen 1998, Viitanen's 1997a). The example case was exfiltration in wood-frame construction in Ottawa. Based on the mold growth index model, the following conclusions may be drawn. Damage analysis showed that air leakage rate had an impact on the wall system performances. The results showed that the mold growth index depends on the indoor relative humidity. Higher indoor relative humidity increases the damage rate. For the present case of exfiltration mold growth will dominate at the bottom of the wall.

The proposed system for durability assessments requires further experimental evidences and verifications. The damage in building envelope is not only due to biological growth, but it could be also due to structural or physical damage. Thus, the

authors have conducted a series of experiments to measure physical damage in wood based products. The results of these experiments would be presented in future publications.

8 Acknowledgements

The authors gratefully acknowledge the technical assistance of Mr. John Lackey in preparing test set-up and samples as well as taking various measurements of the test samples. Useful discussions with Dr. Paul Morris of Forintek Canada Corp. are gratefully acknowledged.

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73 DURABILITY OF WOOD

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Abstract

At present a rapid-train track is built between downtown Stockholm and the international airport of Arlanda. It is partly passing through densely populated suburbs and therefore noise barriers are built along the track. They are about 3 meters in height and built in sections of 4 meters each and the main part is made of larch *Larix sp.* These wood species have a high content of heartwood and is therefore expected to have a higher durability than many other commonly used wood species e.g. pine or spruce. However, the applicability of this is disputed and experimental studies are needed for verification.

In order to compare the deterioration of larch with other species nine of the sections have been replaced with other wood materials with and without additional treatment. Four of these use new, arsenic-free, impregnation systems and two use a well established CCA-based system. In the test series untreated pine and spruce as well as the original larch is also included. All materials have been carefully documented, regarding relevant material properties for the wood and the treatment systems. In the paper an overview of the test set-up is given.

The test has been running for approximately three years and no distinct results can or should be seen so far. The deterioration process will be evaluated on the basis of visual inspections as well as microscopical studies. An evaluation based on the environmental conditions is also planned.

The results are expected to give a good base for judging and comparing different treatment systems, and type of wood. Furthermore, to make it possible to model the deterioration process in the view of the modern SLP concept (Service Life Prediction).

Key words: Wood, Durability, Field test, Traffic noise barrier

1 Introduction

At present, a rapid-train track is built between downtown Stockholm and the international airport of Arlanda. It is partly passing through densely populated suburbs and therefore noise barriers are built along the track. They are built in sections of 4 meters each and the main part is made of larch */Larix sp/*. These wood species have a high content of heartwood and are therefore expected to have a higher durability than many other commonly used wood species e.g. pine or spruce. However, the applicability of this is disputed and experimental studies are needed. (Viitanen, 1997).

In order to compare the deterioration of larch with other species, nine of the sections have been replaced with other wood materials, with and without an additional treatment. All materials have been carefully documented regarding relevant material properties (wood and treatment).

The field test is also an attempt to be a part in a methodological way to predict service life.

2 Principles for service life prediction

A methodological way to predict the service life of building materials and components, is object of standardisation in the ISO draft "Service Life Planning - Part 2: Service Life Prediction Principles" (ISO, 1998), which is partially based on the RILEM Technical Recommendation "Systematic Methodology for Service Life Prediction of Building Materials and Components". (Master and Brandt, 1989).

A prediction according to the standard relies on establishing performance-over-time functions that describe how measured values of some chosen performance characteristic of a material are expected to vary over time. With performance-over-time functions established for the range of in-use conditions considered and agreed performance criteria, all essential elements are known to complete a service life prediction. The methodology includes identification of required information, the selection or development of exposure programs and evaluation methods, exposure and evaluation, interpretation of data, and reporting of results. It employs an iterative approach that permits improved predictions to be made as the base of knowledge grows. (Norén et al, 1998)

The essential steps in a prediction process are shown in Fig 1.

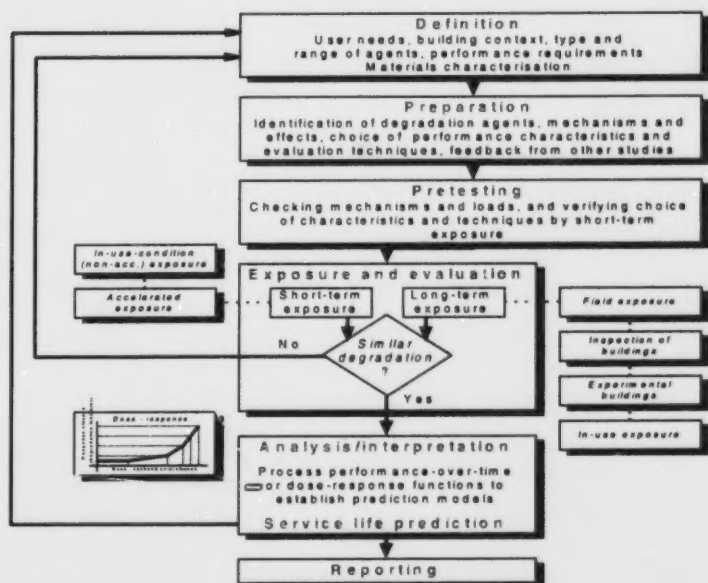


Fig 1: Systematic methodology for Service Life Prediction

3 Description of the noise barrier

The noise barrier is built in sections, each of which has a length of 4 m and a height of about 3 m. The barrier is built with horizontal planks attached by screws to vertical studs. The entire sections are mounted on vertical steel studs with concrete foundations. In order to prevent water from running on and into the structure, horizontal bars with triangular cross section are mounted at a distance of 500-mm. (Paulsen, 1996).

Each section is built in two parts where the lowest part is replaceable. The reason for this is the expected contact with the ground and thus a substantially lower duration compared to the rest of the structure. A sealing strip of EPDM rubber separates the upper and the lower parts. In figure 2, we present a typical cross section of the structure.

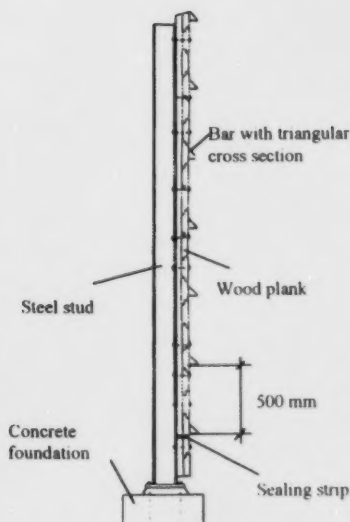


Figure 2: Cross section of the structure.

4. Test material

As alternatives to the original larch the following materials are included in the test.

- Pine /*pinus sylvestris*/ treated with four new products of wood preservatives all in wood preservation class. **NTR/AB*** :
 1. Kemwood ACQ 1900
 2. Kemwood ACQ 1900 with surface treatment
 3. Tanalith MCB
 4. Wolmanit CX-S
 5. Scan IMP
- Pine /*pinus sylvestris*/ treated with CCA (copper, chromium and arsenic) , wood preservation class. **NTR/A***, Kemwood K33 C60
- Pine /*pinus sylvestris*/ treated with CCA (copper, chrome and arsenic) wood preservation class. **NTR/AB***, Injekta K33 C
- Pine /*pinus sylvestris* without additional treatment.
- Spruce /*picea abies*/ without additional treatment.

The declaration of contents for the products of wood preservatives is shown in Table 1.

Table 1: Declaration of content for wood preservatives

Product	Chemical	Content	CAS-number
Kemwood ACQ1900, AB	N-alkylbenzyltrimethylammoniumchloride Tetramincopper	4,8	16828-95-8
		23	61789-71-7
Tanalith MCB, AB	Tebukonazol Boric acid Copper (II) hydroxycarbonate	0,5	107534-96-3
		5,0	10043-35-3
		22,0	12069-69-1
Wolmanit CX-S, AB	Copper HDO Copper (II) hydroxycarbonate Boric acid	6,1	-----
		8,1	12069-69-1
		4,0	10043-35-3
Scan Imp KF, AB	Propiconazole Cocobenzyltrimethylammoniumchloride	5-10	-----
		30-60	61789-71-7
Injekta K33 C, AB	Chromiumtrioxide Diarsenicpentoxide Copper (II)oxide	34,0	1333-82-0
		24,0	7778-39-4
		14,0	1317-39-1
Kemwood K33 C60, A	Chromiumtrioxide Diarsenicpentoxide Copper (II)oxide	27,2	1333-82-0
		19,5	7778-39-4
		10,6	1317-39-1

*Wood preservation Class. A and AB are 2 of 4 grades in the Swedish standard SS 056110(INSTA 140).

Class. A: Full penetration to the heartwood and full retention of preservatives in the sapwood. The field of application is for wood in contact with ground and freshwater and also for wood above ground, which demand extraordinary treatment.

Class AB: Wood used in constructions above ground which mean full penetration to the heartwood and same or less retention as for Class A

5 Documentation of the test sections

For all the planks in the ten test sections the following information is documented:

- The content of heart wood
- The width of the annual rings
- The quantity and length of cracks on the end grain
- The moisture content
-

All test sections have also been strictly documented by photos. The content of heartwood and breadth of the annual rings are estimated by visual inspection.

Length and breadth of cracks on the end grain, longer than 5 cm has been documented.

The moisture content is measured in three points, in the middle and at the end grains of the planks, using a protimeter (Gann hydromette HT 85digital). In laboratory, the retention of additional treatment in the wood is analyzed and the bulk density of the planks has been determined.

Furthermore, 100 screws in the screw union have been weighted, and the average weight, variance and standard deviation are documented.

6 Future measurements and documentation

The test has now been running for approximately three years and no distinct results are given.

The deterioration process will be evaluated based upon visual inspections as well as from microscopical studies.

The visual inspection is divided in two parts:

- Changing in quantity and length of cracks on the end grain.
- The extent of biological degradation

By using microscopical techniques, the type of microorganism attacking the planks can be identified.

The corrosion of screws will be evaluated by measuring the mass loss

Besides measurements and documentation of the test sections, the environment will be documented for an environmental characterization. The data used will be available from a meteorological station close to the noise barrier.

7 Final remarks

The primary aim with the field study is to compare the durability of larch with other sorts of wood e.g. spruce and pine. Secondary is to compare, the efficiency of new arsenic free impregnation systems with well established CCA-systems

Finally is the intention to implicate the study in the service life methodology.

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74 DOCUMENTATION OF CORRECT USE OF WOOD PROTECTION BY DESIGN

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Abstract

This paper describes a research project which documents that correct use of wood protection by design in combination with natural durable wood to some extent can substitute chemical treated wood. The research project consists of both laboratory and full-scale field tests and is conducted by the Danish Technological Institute and the Danish Building Research Institute.

Keywords: Wood protection by design, durability, tests.

1 Introduction

In the European Union (EU) 6 to 6.5 million m³ wood (durability class 4 and class 5, EN 350 and EN 460, see appendix A) is impregnated with chemical agents on a yearly basis. A very significant part of the impregnated wood is used in the construction industry. This wood is primarily treated against biodegradation. However, chemical treated wood constitutes an environmental problem which applies to both production, use and disposal after end use. These problems are dealt with in different ways by the European countries.

In Denmark for instance the awareness of these environmental problems has caused the Danish authorities, Danish Environmental Protection Agency (Danish EPA), to introduce stricter regulations in recent years for wood treated against biodegradation. E.g. has the production and use of arsenic-containing wood inclusive CCA treated wood been prohibited.

Wood is a popular building material because it is friendly to the environment, i.e. wood is CO₂ neutral and renewable. Adding chemicals to preserve wood is in conflict

Durability of Building Materials and Components 8 (Volume One). Edited by M.A. Lacasse and D.J. Vanier. Published in 1999 by NRC Research Press, Ottawa ON K1A 0R6, Canada. ISBN 0-660-17737-4

with this "clean" image but on the other hand a reasonable durability is requested by consumers and the construction industry. Therefore there is a need to develop or rediscover alternative preservation methods.

When used on the exterior of houses and for other outdoor constructions such as noise barriers wood can alternatively be protected by designing the constructions in such a way that the wood does not accumulate moisture. This is called wood protection by design. A combination of using this method and wood with adequate natural durability will decrease the use of wood treated with chemicals.

No complete documentation has yet been prepared on the efficiency of wood protection by design nor on the natural durability of wood above ground. This was confirmed in a literature review *Wood protection by design against biological deterioration - a pre-project* which was carried out by the Danish Technological Institute with support from the Danish EPA in 1995. In the pre-project 1200 references were reviewed showing some documentation but not in a complete form. Following this (1996-1998), a research project co-funded by the Danish EPA, Nordic Industrial Fund/Nordic Wood and the EU to document the efficiency of wood protection by design has been carried out. The project which included both laboratory and full-scale tests was conducted by the Danish Technological Institute and the Danish Building Research Institute. This paper presents the main results of the research project.

2 Identification of design parameters for wood protection by design

The first step of the research project was to identify parameters which have influence on the service life of wood in outdoor constructions. In the selection process the efficiency of the parameter itself, how easy it is to build-in in a construction for both designer and builder and traditional workmanship used in the past were taken into consideration. The following design parameters were selected:

- *Wood species/wood types.* Heartwood of the softwood species with red heartwood such as western red cedar, larch, douglas fir and pine, and heartwood of oak and to some extent spruce are expected to have properties which make them suitable in above ground situations as alternatives to chemical treated wood.
- *Orientation/kerfing.* Turning the inside face (face near the pith of the log) against the weathering and/or kerfing of the wood on the outside face (face further from the pith of the log) is expected to reduce cracks and scratches of the surface.
- *End sealing.* The absorption of water is much faster through the end surfaces than through the side surfaces. Using suitable end sealing products is expected to reduce water absorption.
- *Vertical/horizontal exposure.* Determining which orientation of a siding, horizontal or vertical, provides the longest service life.
- *Surface treatment.* Rainwater combined with sun will cause wood to decay. Using more or less efficient paints, oils and cutting methods

(bandsawing/planning) is expected to make the wood resistant to water intake and decrease the rate of deterioration.

- *Fixing method.* Different methods of fixing wood are used. Nails, screws and gun nails made from different metals, e.g. iron, galvanised iron and stainless steel, have different durability. It is expected that the durability of the fixing also depends on the wood species.
- *Overhang.* Deep overhangs on buildings protect the wood sidings/panels from the rain. Determining the importance of the depth of the overhang in relation to service life will give valuable information to the designer.
- *Design of drip caps.* Different designs of drip caps are used, e.g. above and under windows, some more efficient than others. Determining which parameters are important for designing the drip caps will improve the constructions in relation to service life.

3 Test programme

Documentation of wood and wood constructions' service life demands long term natural exposure but in this project accelerated testing was used due to lack of time. The effect of the design parameters was measured by exposing them to a significantly harder climate than they would normally be exposed to. These tests took place in laboratories. Details and documented test results of the laboratory tests are described in a separate paper, *Protection of wood by design*, by Danish Building Research Institute. However, not all design details were suitable for laboratory tests and the results also had to be verified therefore full-scale field tests with the same test parameters were conducted.

4 Laboratory tests

4.1 Driving rain chamber

4.1.1 Materials and methods

In a driving rain chamber the protecting effect of different types of painted drip caps was tested. The test parameters were: slope, depth and design of the drip notch.

4.1.2 Test results

The test showed that a drip cap is essential but the design of the notch is of little importance. Figure 1 shows the most efficient of the tested profiles.

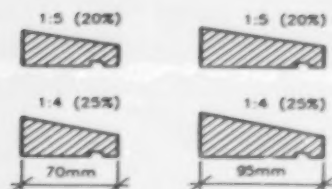


Fig. 1: Drip cap profiles

4.2 "The four seasons", climate simulator

4.2.1 Materials and methods

In "the four seasons", a climate simulator, tests were made to evaluate a number of selected test parameters. The test specimen dimensions were 19 x 85 x 361 mm. The samples were exposed to artificial ageing in the form of cyclic exposure to heat, UV radiation, rain, frost and thaw. Six wood species as well as various surface treatments and orientations etc. were tested. See table 1.

The following test methods were used to evaluate the tested parameters:

- Continuous moisture and temperature measurements of selected test specimens.
- Moisture content based on weight and cupping of all the test specimens were determined at certain intervals.
- Cracks were measured and photographed.
- CT scan (Computerised Axial Tomography).
- Density, growth rings and fungus attack were also taken into consideration when evaluating the test results.

4.2.2 Test results

The results were as follows:

- European larch and WRC (Danish origin) are considered to have the longest service life of the eight wood species tested due to little moisture absorption, maintained geometric stability, and a minimum of cracking.
- Orientation of the test specimens proved to be important, i.e. the inside face showed smaller and less cracks than the outside face. A cracked surface will more easily absorb and accumulate water.
- Kerfing the test specimens resulted in less cupping (maintained geometric stability) and minimised the number of and the width of cracks.
- A CT scan showed that end-sealing minimises moisture accumulation.
- Samples treated with linseed oil and wood oil absorbed slightly less water than samples not treated.
- It was not been possible to establish any difference between the various fixings tested nor determine whether horizontal or vertical exposure give a longer service life.

Table 1: Design parameters, number of tests, and test variables

Design parameter	Specimens	Test variables
Wood species/types	64	Heartwood: European oak, <i>Quercus robur</i> European larch, <i>Larix decidua</i> Douglas fir, <i>Pseudotsuga menziesii</i> WRC (Danish origin), <i>Thuja plicata</i> Scots pine, <i>Pinus silvestris</i> Norway spruce, <i>Picea abies</i> Sapwood: Scots pine, <i>Pinus silvestris</i> Norway spruce, <i>Picea abies</i>
Orientation/kerfing (Scots pine/sapwood)	32	Inside/outside face orientation With or without kerf
End sealing (Scots pine/sapwood)	32	Paints, glues and reference specimens without sealing
Vertical/horizontal exposure (Norway spruce and Scots pine/both sapwood)	32	Vertical/horizontal orientation of panels
Surface treatment (Norway spruce and Scots pine/both sapwood, European oak and WRC [Danish origin]/both heartwood)	64	Bandsawn Planned Reference paint Linseed oil Wood oil
Fixing method (Norway spruce/sapwood)	32	Dowels (wood) Screws (stainless steel) Nails (galvanised iron) Gun nails (stainless steel)
Total	288	

4.3 Large climate simulator

4.3.1 Materials and methods

On the basis of the results from the laboratory tests described in paragraphs 4.1 and 4.2 and to verify these, a full-scale laboratory test was carried out. An exterior wall unit measuring 2900 x 3200 mm with bevel siding made out of larch (European and Siberian origins) and WRC (Danish and Canadian origins) was tested in a large climate simulator. The test specimen (bevel siding) dimensions were 17.5 x 135 x 500-3000 mm. In order to test various window details and ways of building-in windows, two windows (600 x 1200 mm) were fixed in the unit. Table 2 shows design parameters and test variables. The wall unit was tested for six months in climate cycles exposed to heat, UV radiation, rain, frost and thaw.

The following test methods were used to evaluate the tested parameters:

- Continuous moisture and temperature measurements of selected test specimens.
- Moisture content based on weight.

- Cupping was registered on all test specimens.
- Cracks were measured and photographed.
- Gaps between the sidings.
- Water repellence was tested.
- Density, growth rings, fungus attack and a general visual evaluation were also taken into consideration when evaluating the test results.

Table 2: Design parameters and test variables

Design parameter	Test variables
Wood species/types	Heartwood: Larch (European origin), <i>Larix decidua</i> Larch (Siberian origin), <i>Larix siberica</i> WRC (Danish origin), <i>Thuja plicata</i> WRC (Canadian origin, virgin wood), <i>Thuja plicata</i>
Orientation	Inside/outside face orientation
Surface treatment	No treatment Wood oil
Fixing method	Nails (galvanised iron) Gun nails (stainless steel)
Fixing distance from cross-cut end of siding	20 mm 50 mm 100 mm 150 mm
Window details	Drip caps Joints Etc.

4.3.2 Test results

The results were as follows:

- The tests showed no significant difference between the wood species with regard to service life: showed same percentage of moisture absorption, and equal geometric stability and magnitude of cracks.
- Turning the inside face against the weathering versus the outside face resulted in smaller and less cracks. However, this was not significant.
- The bevel siding treated with wood oil proved to be water repellent compared to the non-treated siding.
- It has not been possible to establish any difference between the various fixings tested.
- A fixing distance from the cross-cut end of siding of 20 mm showed damaging cracks. A distance of 50 mm and more did not show cracks of significance.
- The design of the window details tested proved to be adequate.

5 Full-scale field tests

5.1 Test wall

5.1.1 Materials and methods

On the basis of the results from the laboratory tests described in paragraph 4 and to verify these a full-scale field test was carried out on Danish Technological Institute's field test area (12° east, 55° north). A rack with two walls measuring 2500 x 15000 mm facing north and south was constructed in an open outdoor space. The test specimen (panel) dimensions were 25 x 100 x 1200-1500 mm. Table 3 shows design parameters and test variables. The walls were finished March 1, 1998, and test readings will continue as long as they provide valuable information.

The following test methods are/will be used to evaluate the tested parameters:

- Continuous moisture and temperature measurements of selected test specimens.
- Moisture content based on weight.
- Cup and crack registration.
- Photography.
- Gaps between the sidings.
- Water repellence.
- CT scan.
- Density, growth rings, fungus attack and a general visual evaluation will also be taken into consideration when evaluating the test results.

5.1.2 Test results

The results are few and preliminary due to the short test period. However, there is already a very clear effect of the overhang.

Where surface treatment has been used and there is no overhang the natural wood colour is preserved. And where there is neither overhang nor surface treatment the wood has turned grey. Corroded galvanised iron fixings have caused rain water run-off stains, especially on the WRC. Blue stains have been observed on sapwood of Norway spruce and Scots pine.

5.2 Noise barriers

Based on the results from the laboratory tests the Danish Road Directorate has been advised in connection with the construction of wooden noise barriers to use non-impregnated wood and wood protection by design. Subsequently, approximately 8 km wooden noise barriers have been constructed at various locations in 1997-98 and more are on the drawing board. The Danish Technological Institute has been involved in the construction of all these noise barriers and the results from the research project have been incorporated. However, some of the noise barriers are used as real life situation field tests as different wood species, end sealing products, surface treatments, and fixing methods have been used in the different noise barriers in order to prove which are the most suitable for the purpose. Examples are given below.

Table 3: Design parameters and test variables

Design parameter	Test variables
Wood species/types	<i>Heartwood:</i> Larch (European origin), <i>Larix decidua</i> Larch (Siberian origin), <i>Larix siberica</i> Douglas fir, <i>Pseudotsuga menziesii</i> WRC (Danish origin), <i>Thuja plicata</i> WRC (Canadian origin, virgin wood), <i>Thuja plicata</i> Scots pine, <i>Pinus silvestris</i> Norway spruce, <i>Picea abies</i> <i>Sapwood:</i> Scots pine, <i>Pinus silvestris</i> Norway spruce, <i>Picea abies</i> <i>Reference:</i> Chemical treated Scots pine (commercial)
Orientation/kerfing	Inside/outside face orientation With or without kerf
End sealing	Paints, glues and reference specimens without sealing
Vertical/horizontal exposure	Vertical/horizontal orientation of panels
Surface treatment	No treatment Wood oil
Fixing method	Screws (stainless steel) Nails (galvanised iron) Gun nails (stainless steel)
Fixing distance from cross-cut end of siding	20 mm 50 mm 100 mm 150 mm
Overhang	None 500 mm 1000 mm

5.2.1 Budding noise barrier

The noise barrier is built on freeway 3 near Copenhagen and measures 1180 m with a height of 2.7 m. The wood species used are: European larch, Norway spruce, European oak, WRC (Danish origin) and douglas fir. The siding is both horizontal and vertical, a glue is used as end sealing, surface treated with wood oil and stainless gun nails are used as fixings.

5.2.2 Naestved noise barrier

The noise barrier is built on a freeway 100 km from Copenhagen near Naestved and measures 1420 m with a height of 2.8 m. The materials used are: larch (European) and chemical treated wood (Scots pine). It is surface treated with a coloured wood oil, a glue is used as end sealing and galvanised gun nails are used as fixings.

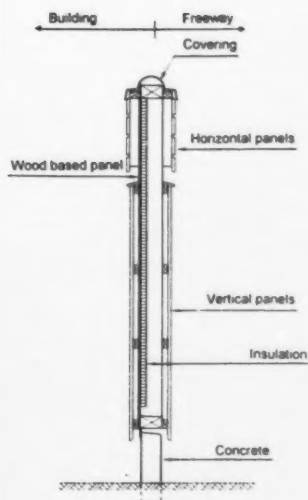


Fig. 2: Buddinge noise barrier

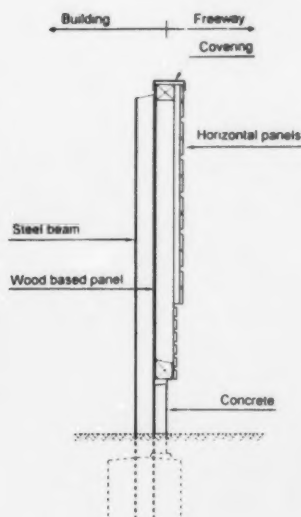


Fig. 3: Naestved noise barrier

6 Concluding remarks

The test results show that it is possible to some extent to substitute chemical treated wood with a combination of wood with adequate natural durability and wood protection by design. The field tests will in the future give more reliable and more powerful information to be used by designers and builders. But there is a need for education of builders and designers in this area to upgrade wooden outdoor constructions.

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Appendix A:

Natural durability to wood-destroying fungi according to EN 350 and EN 460

Wood-destroying fungi - Guidance on the durability classes of wood species for use in hazard classes

Hazard class	Durability class				
	1 Very durable	2 Durable	3 Moderately durable	4 Slightly durable	5 Not durable
1 Above ground, covered (dry)	o	o	o	o	o
2 Above ground, covered (risk of wetting)	o	o	o	(o)	(o)
3 Above ground, not covered	o	o	(o)	(o) - (x)	(o) - (x)
4 In contact with ground or fresh water	o	(o)	(x)	x	x
5 In salt water	o	(x)	(x)	x	x

Key

- o Natural durability sufficient.
- (o) Natural durability is normally sufficient, but for certain end uses treatment may be advisable.
- (o) - (x) Natural durability may be sufficient, but depending on the wood species, its permeability and end use, preservative treatment may be necessary.
- (x) Preservative treatment is normally advisable, but for certain end uses natural durability may be sufficient.
- x Preservative treatment necessary.

Note: Sapwood of all wood species should be regarded as durability class 5.



PART FOUR

**DURABILITY OF
POLYMER-BASED MATERIALS**



75 DEVELOPMENT OF A DURABILITY CLASSIFICATION SYSTEM FOR PROTECTIVE COATINGS FOR CONCRETE

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Abstract

Anti-carbonation coatings for concrete act as barriers to carbon dioxide and thus prevent carbonation. They are extensively used in the rehabilitation of concrete structures throughout the world to prolong the service life of a structure. This paper describes a project, which aims to evaluate the performance of coatings after natural and artificial weathering. Test methods currently available in draft and current CEN Standards (crack bridging, water vapour resistance, CO₂ diffusion resistance and liquid water resistance) will be used. The duration of the programme will be five years for natural weathering and a minimum of 2500 hours QUV exposure for artificial weathering.

The project started in April 1997 and should be completed in March 2003. Ten different coating systems have been applied to different strength concrete beams, mortar specimens and unglazed ceramic tiles. Urban and marine sites are being used to naturally weather the specimens. The coatings will be monitored during the period of weathering to investigate changes in the physical properties of the coating. From this information a classification system will be evolved to guide the specifier in the selection of a coating to suit their purpose. The work will also be used to validate European Standard tests and classification systems currently being drafted. This paper summarises the work to-date and background information on the project.

Keywords: coatings, concrete, carbon dioxide, resistance, diffusion, permeability, water vapour

1 Introduction

Problems associated with carbonation of concrete have been thoroughly documented (Building Research Establishment 1982, Everett, L.H. et al, 1980, Concrete Society, London 1984, Currie, R.J 1986, Rothwell G.W. 1985). Carbonation may result in corrosion of depassivated reinforcement, and deterioration and loss of serviceability of structures. Surface coatings and treatments are increasingly considered as an integral part of the repair and maintenance process to reduce the rate of further deterioration (CIRIA Technical Note 130, 1987).

Surface treatments to protect concrete must have certain key of properties to be effective and durable. Anti-carbonation coatings are surface treatments that have low permeability to carbon dioxide and reduce the risk of carbonation. Typical properties that a surface treatment needs to prevent carbonation are:

- a dry film thickness of at least 200 μm , in order to cover effectively a rough concrete surface and a defect free film
- good adherence
- ability to accommodate anticipated movements in concrete; this is specified as the crack-bridging ability of a coating.
- good colour retention, chalking and ultra-violet (UV) resistance
- a high liquid water resistance
- a low water vapour resistance
- a high resistance to carbon dioxide
- a service life of at least 10 years

In future the selection of coatings for concrete in Europe will be specified by European Standards. The aim of this project is to evaluate the performance of anti-carbonation coatings and to see whether the test methods, proposed classes and classification system in the European standards are appropriate for selecting anti-carbonation coatings.

2 European standards and future selection of coatings for concrete

A fundamental goal of the European Union (EU) is to remove barriers to trade among member states. Replacement of national standards by European Standards is a key element in the single market policy. All members of the EU have signed the Construction Products Directive (CPD), which has led to the development of European Standards for the construction industry. In the next two to three years, national standards in member states of the EU for surface treatment of concrete will be replaced by European Standards.

These developments have serious implications for UK industry's approach to standards. It is important to realise that the Single Market requires a culture change in the way standards are used. An important consequence of European Standards is that public bodies are required to specify wherever possible to European standards for all construction products and this include coatings for concrete, i.e. European Standards are de-facto mandatory for public or privatised utilities if applicable.

Most of the European Standards and Classification Systems for coatings on concrete will be in force within the next three years. Of particular interest to the industry will be draft EN 1062-1 (Classification for Masonry Paints), draft EN 1504-2 (Classification System for Surface Treatment of Concrete), ENV 1504-9 (Guidance note on the principles of protection and repair of concrete) and draft EN 1504-10 (Site application of products and systems and quality control of the works).

Surface treatments for concrete with a maximum dry film thickness of 5 mm are covered by draft EN 1504-2. This standard will give guidance on the selection of coatings for concrete and the required mandatory tests for properties of coatings. For example, if a coating is to be used for ingress control, the manufacturer will have to test the coating for adhesion (pull-off test) and carbon dioxide, liquid water and water vapour permeabilities.

3 Materials, specimen preparation and exposure

Ten commonly used commercial coating materials were selected. The coatings were applied to unglazed ceramic tiles and prisms of concrete (100 x 100 x 750 mm) with 20 N and 33 N strength. The ceramic tiles were selected for their high porosity, low resistance to carbon dioxide and their low relative cost. Specimens will be naturally aged on the south coast of England and at BRE for 5 years, and artificially weathered in a QUV machine for at least 2500 hours. The specimens will be removed periodically during the weathering programs and the tested for the following properties:

- Colour and gloss of coatings
- Carbon dioxide permeability of coatings
- Water vapour permeability
- Adhesion of coatings to concrete
- Carbonation depths of concrete
- Crack-bridging ability of coatings

The initial tests on the specimens i.e. time zero, will be completed by the end of November 1998, and the weathering programs will then start. The tests used in the program and their backgrounds to the test are described below.

4 Carbon dioxide permeability

Systems for measuring the carbon dioxide resistance of coatings can be classified according to the type of test cell used. There are two types available; the absorption cup (or gravimetric method) and systems based on the diffusion cell. Those based on the cup method rely on the absorption of CO₂ in the cup. Absorption is measured by weighing the cup regularly to determine the increase in weight against time. Diffusion cell systems measure the quantity of CO₂ passing through the membrane test by analysis of the gas stream.

A gravimetric test developed at BRE has shown that the test of carbonation of concrete may be reduced by a number of different coatings and surface treatments (Rothwell, G.W. et al, 1989).

Further work has shown however, that coatings could become less effective carbon dioxide barriers over a period of 2000 hours artificial weathering (Wang Wei, et al, 1990), that the reduction of barrier properties varied according to the chemical type of binder used in the coating, and that the reduction was more significant for some coatings than for others.

Klopfer and Engelfried (Engelfried, R, 1977, Engelfried, R, 1983) also developed a gravimetric method of measuring resistance of coatings to carbon dioxide. They tested resistance on paper substrates, which are not suitable either for natural or for artificial weathering. The gravimetric method developed at BRE uses cementitious substrates and is destructive. It has limited sensitivity and is not suitable for measuring carbon dioxide resistance of high performance coatings. To overcome the limitations of the gravimetric test a non-destructive method of test using an isostatic partition cell has been developed.

The changes in performance of a coating produced by accelerated weathering need to be confirmed by measuring performance over a period of natural exposure. Natural exposure test data are also needed to determine the relationship between accelerated test results and likely in-service performance. This will allow an accelerated exposure regime for the prediction of carbon dioxide resistance of coatings to be defined.

The measurement of carbon dioxide resistance of coatings has been carried out for over 15 years in the UK, Denmark and Germany, but the methods used and the results obtained are not widely understood.

4.1 Apparatus

Development of partition cell apparatus

Thomas and Gent (Thomas, A.M., et al 1945) used a partition cell to study permeation and sorption of moisture vapour in varnish films. A similar method was employed to measure the permeability of plastic films for packaging applications (Davis, W.W., 1946). Pasternak, Schimscheiner and Heller (Pasternak, R.A. et al, 1970) first proposed the use of a partition cell coupled with a gas chromatograph to determine the permeability of films to gas mixtures, and this was further developed by Pye, Hahn and Panar (Pye, D.G., et al, 1976).

Sherwood (Sherwood, A.F., et al, 1975) studied the carbon dioxide resistance of coatings using a partition cell. The quantity of carbon dioxide passing through the film was measured by gravimetric means, a similar approach to that later employed by Engelfried and Klopfer (Engelfried, R., 1983, Thomas, A.M. et al, 1945, Davis, W.W., 1946), whose method is derived from the DIN, EM and ASTM tests for measuring water vapour permeability of polymer films (Draft EN 1062-1, ASTM E96-80, 1980, DIN 53 122, 1974). These tests employ the diffusion cell first proposed by Deeg and Frosch (Deeg, G et al, 1944). Brianza and Piguet (Brianza, M.A., et al, 1986) have also developed a test method similar to the BRE mortar block test (Rothwell, G.W et al, 1989, Wang Wei, et al, 1990).

Several others have developed apparatus based on the partition cell, Möller (Möller, L., 1985) used a partition cell with gravimetric determination derived from Sherwood's report. Others employed instrumental techniques to measure the rate of diffusion of carbon dioxide through the test sample. Désor and Pauly (Désor, U et al, 1985, Désor, U., 1985) used infrared analysis to determine the rate of carbon dioxide diffusion through coatings. Robinson (Robinson, H., 1986, Robinson, H., 1987) used a gas chromatograph to measure diffusion rates of oxygen and carbon dioxide using dry gas mixtures. The equipment employed by these three is all derived from that reported by Davis for testing the carbon dioxide resistance of packaging materials (Davis, W.W., 1946). The present study was carried out using a similar system of partition cells and gas chromatographic analysis. The method of analysis by Durbeck and Niehaus (Durbeck, H.W et al, 1978), Gates et al (Gates, W., et al, 1981), Kern and Elser (Kern, H., et al, 1978) and Latif et al (Latif, S., et al, 1983). These data indicated that mixtures of gases containing water vapour could be analysed successfully by gas chromatography.

4.2 BRE Carbon dioxide permeability test

The BRE test methods is based on a partition cell see Figure 1. The apparatus has three main parts (Figure 2). A gas handling system provides a stream of a mixed, dried test gas and a helium stream, which are linked to opposite sides of a differential mercury manometer. The pressure in the two streams is equalised. The test stream is supplied through a manifold to the lower chamber of each cell, where it may be humidified. The test gas is a mixture of 15% carbon dioxide in nitrogen. Helium is supplied at a pre-set flow rate to the upper chamber of each cell via a similar manifold. The manifolds also act as ballast volumes to smooth minor fluctuations in the flow rates through the cells.

The coating under test is supported on a porous ceramic tile to form the test membrane. A plastic ring clamps the membrane against an 'O' ring mounted in a rebate in the wall of the upper chamber, forming a seal and defining the area of coating under test. The pressure balance ensures that the two chambers of each cell contain different partial pressures of gases at the same overall pressure.

Gas transport through the membrane occurs by diffusion. Gas diffusing through the membrane from the lower (test gas) chamber enters the sweep-stream in the upper chamber. At equilibrium the rates of diffusion of the component gases in the test mixture and of helium through the membrane reach a steady state and the test gas content of the helium sweep-stream reaches a constant value.

A selection valve connects the cells to the sampling valve of the gas. The sweep gas of each cell is sampled in turn followed by the helium and test streams. The cycle is repeated ten times. Typically the steady-state condition is reached in about eight hours and the data processor controlling the apparatus is programmed to analyse the gases after this period. Examination of the data after the analytical cycle is complete indicates when equilibrium was attained, and any analyses carried out before equilibration was achieved is discounted and the flux calculated for the remaining data. A plot of the GC detector response against time give a series of gaussian peaks, each of which correspond to one component of the test gas mixture under ideal conditions. A digital integrator gives values of peak area,

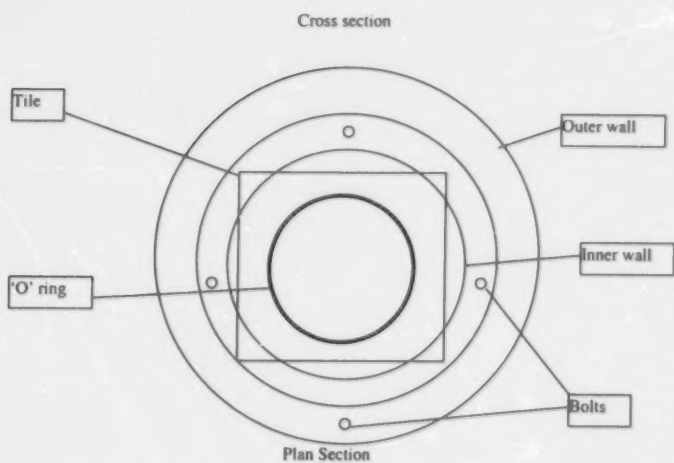
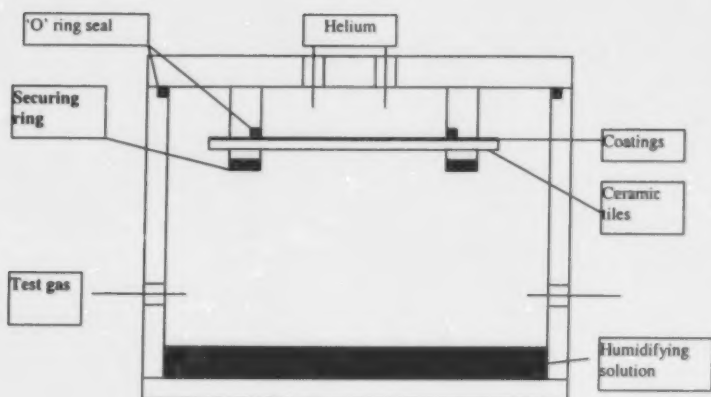


Fig. 1: Schematic diagram of partition cell

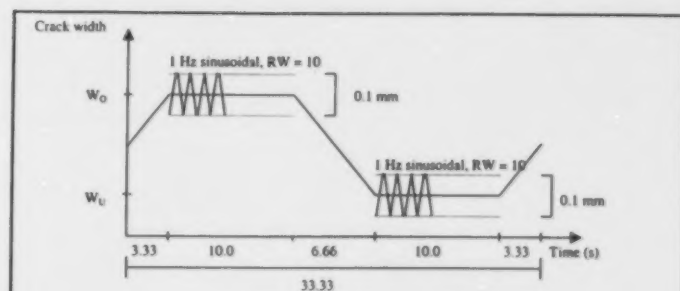


Fig 2a: Change of crack width as a function of time for classes B2.2 and B3.2. 1000 crack cycles as a trapezoidal function using 0.03 Hz, with superimposed crack cycles as a sinusoidal function using 1 Hz

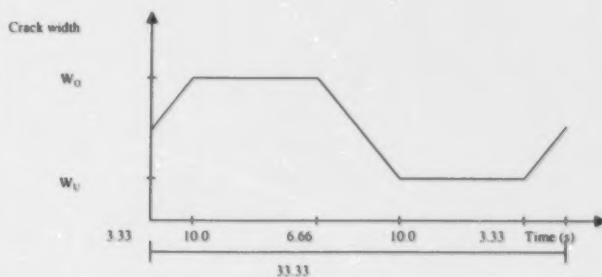


Fig 2b Change of crack width as a function of time for classes B0, B1, B2.1 and B3.1. 1000 crack cycles as a trapezoidal function using 0.03 Hz

Fig. 2: Crack width profiles for the draft European dynamic crack-bridging test

height and retention time. The peak area is used to determine the concentration of carbon dioxide in the samples from the partition cells.

Given the volume of sample injected, the flow rate of the sweep helium through the partition cell and the concentration of CO_2 in the sample the quantity of carbon dioxide passing through the specimen per minute may be calculated, thus:

$$Q = \frac{F \cdot C \cdot v \cdot M}{V} 10^{-6} \text{ g min}^{-1} \quad (1)$$

where: F = flow rate of helium sweep stream in $\text{cm}^3 \text{ s}^{-1}$

C = concentration of CO_2 in sample

v = sample volume in cm^3

M = relative molecular mass of CO_2

V = molar gas volume at 200°C

Q = quantity of CO_2 in g min^{-1} passing through the coating at steady state

From this value it is possible to calculate the flux, J_{CO_2} , in $\text{g m}^{-2} \text{ day}^{-1}$:

$$J_{\text{CO}_2} = \frac{Q \cdot 1440}{A} \text{ gm}^{-2} \text{ day}^{-1} \quad (2)$$

where A = area of coating in m^2

4.2.1 Measurement and calculation of carbon dioxide resistance

Carbon dioxide resistance may be calculated and quoted in several ways. The terms and units most commonly used are: flux ($\text{g m}^{-2} \text{ day}^{-1}$); diffusion coefficient ($\text{m}^2 \text{ s}^{-1}$); diffusion resistance coefficient (dimensionless) and equivalent air layer thickness (m).

The derivation and calculation of these values is as follows: the flux is a function of the concentration gradient across the coating and of the thickness of the coating. Applying Fick's second law of diffusion, it is possible to determine the coefficient of diffusion for a given thickness of coating, D_{Coating} , from the flux and concentration difference of CO_2 across the coating, thus:

$$D_{\text{Coating}} = \frac{J_{\text{CO}_2} \cdot s}{[c]} \quad (3)$$

where: J_{CO_2} = flux ($\text{gm}^{-2} \text{ s}^{-1}$)

s = thickness (m)

$[c]$ = concentration difference (gm^{-3})

This value has units ($\text{m}^2 \text{ s}^{-1}$), and is therefore not a dimensionless coefficient. It is converted to a dimensionless diffusion resistance coefficient, μ , which compares the diffusion of CO_2 through the coating with the diffusion of CO_2 through still air under the same conditions and is independent of applied coating thickness. It is calculated as follows:

$$\mu = \frac{D_{\text{CO}_2}}{D_{\text{Coating}}} \quad (4)$$

where D_{CO_2} = diffusion coefficient of CO_2 through air ($\text{m}^2 \text{ s}^{-1}$)

Typical values for the diffusion resistance coefficient are in the range 10^6 to 10^8 for surface coatings with a high resistance to CO_2 . This coefficient is used to calculate the effectiveness of the coating in terms of the equivalent air layer thickness, $R_{(\text{CO}_2)}$. The equivalent air layer thickness equates the barrier property of a coating to the thickness of still air with the same diffusion resistance. It is obtained by multiplying the diffusion resistance coefficient, $\therefore$, by the coating thickness, s , as follows:

$$R_{\text{CO}_2} = \mu \cdot s \quad (5)$$

Typical values for $R_{(\text{CO}_2)}$ are between 1 m and 500 m, but higher values are not unknown.

4.2.1 Effects of relative humidity on carbon dioxide resistance

Research at BRE (unpublished) has shown that there are some variations in the values obtained from measurements carried out at the two humidities of test gas, but the general trends observed are similar at both humidities.

The humidity at which the test is conducted does not have as much effect on the measured carbon dioxide resistance as the type of chemical binder used in the coating or the duration of exposure of the coating. In practice, coatings are applied in conditions where they will almost invariably be exposed to moist air. It would therefore be prudent, where possible, to carry out the measurement of carbon dioxide resistance using a humidified test gas.

4.3 Gravimetric method for measuring carbon dioxide permeability

The gravimetric method can only be carried out with humidified test gas if the humidity of the whole test apparatus is maintained at a uniform level. This may prove difficult with the gravimetric method based on the Payne Cup tests (Draft EN 1062-2, ASTM E96-80, 1980, DIN 53 122, 1974). These usually employ an absorbent solid on one side of the coating, and these absorbents are usually absorb water as well as carbon dioxide.

5 Water vapour permeability test

5.1 BS 3177:1959 Cup method for determining water permeability

BS 3177 employs an aluminium dish, commonly called a "Payne Cup". The depth of the aluminium dish is dictated by the permeability of the paint system to be tested the lower the permeability the shallower the dish. The largest dish size recommended by the standard is usually chosen, which gives a test area of 50 cm^2 . Calcium chloride is used as the desiccant. A mechanical or wax seal used to seal the specimens. In the test program a mechanical seal is used, so that it is a non-destructive test. The test is usually undertaken at 23°C and $55 \pm 5 \text{ \%RH}$.

EN 1062-1 uses a Payne cup for the determination of the permeability of paints, as specified in ISO 7782. However, in draft EN 1062-1 the humidity gradient across the coating is obtained by sealing a humidifying solution into the cup, not a desiccant. Also, the coatings are subjected to a weathering cycle prior to testing. Unpublished result at BRE has shown that the results obtained using the two methods are similar.

6 Crack-bridging test

A crack-bridging test is undertaken to determine a coating's ability to accommodate movement of cracks in concrete due to load and temperature changes. There are two types of test, static and dynamic. The static test is a one-cycle test, which will give an indication of the maximum crack width that can be accommodated by a coating. The dynamic test will give information about the performance of a coating over an active crack.

6.1 Requirements of the draft European standard (1062-7)

6.1.1 Dynamic crack-bridging test

The latest draft requires that tests be carried out in triplicate. It does not state the design of the test specimen. The draft specifies that the mortar for the specimens for the static and dynamic crack-bridging test should be in accordance with EN 196-1 as follows:

OPC	4.5 kg
Water	2.25 kg
Sand	13.5 kg

For the dynamic test the proposed crack displacement cycle should follow the movements shown in Figure 2 and the requirements for the different classes are given in Table 1. Classes B0 and B1 are for coatings with a minimal requirement for crack bridging, such as tanking coatings. Class B2.1 is for coatings on structures which may be subjected to crack movements due to thermal changes and loading, but not mechanical loading. Class B2.2 is for coatings with the same requirements as class B2.1, but subjected to mechanical loads, such as liquid applied waterproofing systems on bridges, under asphalt. Class B3.3 is for coatings with the

same requirements as for class B2.2, but with a large crack movement requirement and is for coatings subjected to direct traffic as in a car park or warehouse. The test is undertaken at one of the following temperatures; 23°C, 10°C, 0°C, -10°C or -20°C. The test temperature selected should be the most appropriate to the expected service conditions. The proposed pass criterion for the dynamic crack-bridging test is that the coating should remain intact after the test.

Table 2: Draft European Standards (prEN 1062-7) test conditions for different Dynamic crack-bridging requirements for surface treatments

Class	Test conditions (see Figure1)
B 0	$w_o = 0.15 \text{ mm}$ $w_u = 0.10 \text{ mm}$ trapezoid $RW = 100$ $f = 0.01 \text{ Hz}$ $W = 0.05 \text{ mm}$
B 1	$w_o = 0.15 \text{ mm}$ $w_u = 0.10 \text{ mm}$ trapezoid $RW = 100$ $f = 0.01 \text{ Hz}$ $W = 0.05 \text{ mm}$
B 2.1	$w_o = 0.25 \text{ mm}$ $w_u = 0.10 \text{ mm}$ trapezoid $RW = 1000$ $f = 0.03 \text{ Hz}$ $W = 0.15 \text{ mm}$
B 2.2	As above, and $w_L = \forall 0.05 \text{ mm}$ sinusoidal $RW = 20,000$ $f = 1 \text{ Hz}$
B 3.1	$w_o = 0.50 \text{ mm}$ $w_u = 0.20 \text{ mm}$ trapezoid $RW = 1000$ $f = 0.03 \text{ Hz}$ $W = 0.30 \text{ mm}$
B 3.2	As above, and $w_L = \forall 0.05 \text{ mm}$ sinusoidal $RW = 20,000$ $f = 1 \text{ Hz}$

Key:

f = frequency	w_L = super imposed crack movement
RW = number of crack cycles	w_o = maximum crack width
W = change in crack width	w_u = minimum crack width

6.1.2 Static crack-bridging test

The crack displacement rate for the static crack-bridging test should be 0.1 or 0.5 mm per minute. The rate of 0.5 mm per minute is to be used when the static crack-bridging ability of coating is likely to be known to be greater than 3 mm, in order to reduce the time spent testing. The crack shall be induced in the test

specimen at 23°C prior to testing. Four classes have been agreed for the static crack-bridging test and are listed below.

	Temperature (°C)	Crack Width (µm)
Class 1	23	>100
Class 2	-10	>250
Class 3	-10	>1250
Class 4	-10	>2500

7 Conclusion

The work will also be used to validate European Standard tests and classification systems currently being drafted. This paper summarises the work to-date and background information on the project. Data on the testing of the coating will be presented at the conference.

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76 CORRELATION OF THE EFFECTS OF ARTIFICIAL AND NATURAL WEATHERING AT ELASTOMERIC BUILDING SEALANTS

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Abstract

An extensive and comprehensive multi-year durability study was undertaken in which a series of high-performance sealant products were artificially and naturally aged in an effort to determine test methods and test regimes most likely to simulate in service conditions. Sixteen elastomeric products were evaluated of which fourteen were sealants based on either polysulphide, polyurethane or silicone compounds. Artificial ageing was conducted using different weathering apparatus and consisted of various combinations of exposure to fluorescent or Xenon lamp Ultra violet radiation, simultaneous heat ageing at temperatures ranging from 60 to 90°C and, water condensation. Another method was heat ageing in ovens at temperatures up to 140°C. The change in materials properties was characterised through mechanical tests (tensile strength and elongation at break), thermoanalysis and chemical spectroscopy including infrared and photoacoustic spectroscopy. Results of specimens aged artificially are compared with those aged from 5 years open air weathering in Leipzig. The differences between results obtained from open-air weathering and those produced by indoor storage are, for the test periods of five years used up to now, usually slight. Nearly all materials stood up well to artificial ageing at temperatures of between 60°C and 120°C. Correlation existed between artificial and natural ageing for all 16 elastomers tested. The frequency of cases of correlation was scarcely reduced by increasing the testing temperature during the temperature/time period used. Changes of test environment, in most cases, have more effect than mere heat storage. Further tasks include: determination of the causes of non-ageing reactions of certain sealants and the temperature sensitivity of some substrates, along with the experimental classification of such influencing factors on artificial ageing at high temperatures as movement and air contamination.

Keywords: accelerated ageing, correlation, elastomeric sealants, free films, joint models, natural weathering, time compression factor

1 Introduction

There exist a number of problems associated with developing useful tests to assess the long-term performance of sealants. The authors have already produced several reports on the long-term durability of sealants. Empirically-based investigation into this durability was reported on recently, for example, at the IABSE Colloquium Saving Buildings in Central and Eastern Europe, held in Berlin (Bolte and Boettger 1998). With a view to determining long-term durability with regard to artificial ageing techniques, interim reports were published in 1996 (Boettger and Bolte 1996) and 1998 (Boettger and Bolte 1998). The intention here is to report on the current situation.

2 Experimental

2.1 Materials

The elastomeric sealants evaluated in this study are given in Table 1.

Table 1: Material types, curing systems and colours

No.	Material type	Curing system	Colour
SI 5	Silicone rubber	1-part neutral cure, Oxime	grey
SI 6		1-part basic cure, Amine-Oxime	grey
SI 7		1-part neutral cure, Oxime	white
SI 8		1-part neutral cure, Oxime	white
SI 17		1-part neutral cure, Alkoxy (Titanate)	white
SI 18		1-part neutral cure, Benzamide (Titanate)	grey
SI 19		1-part neutral cure, Benzamide	white
SI 21		1-part acid cure, Acetoxy	white
PS 9	Polysulphide rubber	2-part Manganese dioxide cure	grey
PS 10		2-part Manganese dioxide cure	grey
PS 12		2-part Manganese dioxide cure	black
PS 22		2-part Lead dioxide cure	grey
PS 28		prefabricated	grey
PU 23	Polyurethane	1-part	grey
CR 24	Polychloroprene rubber	heat vulcanisation	black
E 25	EPDM rubber	heat vulcanisation	black

2.2 Test methods

2.2.1 Mechanical tests

Prior to testing, all samples were stored in standard conditions of 23 °C and 50% RH until the weight was constant. Following this, three strips were cut from specimens of 45mm length and tested in tension according to ISO 37. Both tensile strength and elongation at break were recorded for each of the specimens. Tensile tests on model joint compounds was carried out using ISO 8339 and

provided information regarding the tensile strength and elongation at break and the tensile modulus.

2.2.2 Physical chemical tests

The glass transition temperature of weathered samples was determined using a dynamic differential scanning calorimeter (DSC), model DSC 200 (Netzsch Geraetebau). Chemical changes at the surface of the weathered samples were examined using attenuated total reflectance (ATR) Fourier transform infrared (FTIR) spectroscopy (IMPACT 400, Nicolet). As well, photo acoustic (PAS) FTIR spectroscopy was used to examine the surface of samples that had a severely degraded surface since this technique readily lends itself to chemical analysis in instances where surfaces are sufficiently deteriorated as to render the ATR method useless. For the PAS technique, a MTEC model 300 photoacoustic cell was used to collect the requisite data.

2.3 Test series

2.3.1 Ageing using fluorescent UV radiation, heat and water condensation (free films)

- 8hrs UV at +60°C; 4hrs condensation at +50°C
 - 8hrs UV at +70°C; 4hrs condensation at +50°C
 - 8hrs UV at +80°C; 4hrs condensation at +50°C
 - 8hrs UV at +90°C; 4hrs condensation at +50°C
- UVCON device, Atlas Co. with UVA 340 lamps 1.000, 2.000, 3.000 6.000hrs

2.3.2 Ageing using XENON-arc radiation, heat and water storage (free films)

- x (3 days +80°C, 1 day water storage, 2 days +80°C, 1 day water storage)
- 21 day UV with SUNTEST CPS unit Heraeus Co. with Xenon arc lamp NXE 15001.000, 2.000, 3.000 6.000 hrs

2.3.3 Thermal ageing (free films)

- +60°C / 10 changes of air per hour
 - +70°C / 10 changes of air per hour
 - +80°C / 10 changes of air per hour
 - +90°C / 10 changes of air per hour
- collective storage in ovens 1.000, 2.000, 3.000 6.000hrs
- +100°C, +110°C, +120°C, +140°C with shortened test times

2.3.4 Artificial ageing combined with extension and compression

Three of 16 materials were examined. The sealants SI5 and SI6 have been found to be especially ageing resistant during the previous tests. On these sealants there have always been cohesion breakdowns, i.e. the adhesion on concrete was very good. The sealant PU23 is the weakest material where ageing is concerned (false batch), although it also has very good adhesion on concrete. Natural weathering and artificial ageing was carried out on joint models, prepared in accordance with ISO 8339. The following combinations apply:

- Natural weathering without and with continuous extension of +25%
Current status: 5 years open air weathering.
- Natural weathering with 1 extension/compression cycle per year, alternately 6 months extension of +12,5% during the winter months and 6 months compression of -12,5% in the summer months
Current status: 1 year open air weathering.
- Natural weathering with 6 extension/compression cycles per year, alternately 1 month extension of +12,5% and 1 month compression of -12,5%
Current status: 1 year open air weathering.
- Artificial ageing (UVCON 80/50) and extension/compression cycles, separately (alternating), after every 1.000 hrs at -20°C extension of +12,5% and at +70°C compression of -12,5% in accordance with ISO 9047
Current status: Completed with 6.000 hrs.
- Artificial ageing (UVCON 60/50 and 70/50) and extension/compression cycles, simultaneously (parallel), at intervals of 2 weeks each, extension of +12,5% and compression of -12,5%
Current status: Completed with 6.000hrs.

2.3.5 Natural weathering and room storage

Storage took place at the site of Leipzig in a temperate industrial climate and in the dark, at room temperature. Start in January 1993.

3 Results

3.1 Tensile tests on free films

3.1.1 Idealised correlation

A basis for interpreting the results may be provided by presenting the idealized correlation expected when comparing naturally aged specimens to those aged artificially. Figure 1 provides the change in elongation at break of an elastomeric material in relation to the ageing temperature and ageing period and compares this with similar changes in elongation for the material aged in natural conditions. The figure does not represent actual data but an idealized representation of the change in elongation at break for specimens aged up to 12.000hrs. These functions are based on the Arrhenius relationship as applied to the selected temperature range. In this case, a 10°C rise in ageing temperature will halve the time to rupture of the elastomer. Depicted in the Figure 1 is an example of a life expectancy of 8 years that, if subjected to artificial ageing at more elevated temperatures, would, on average, fail in the following time frame as described in Table 2.

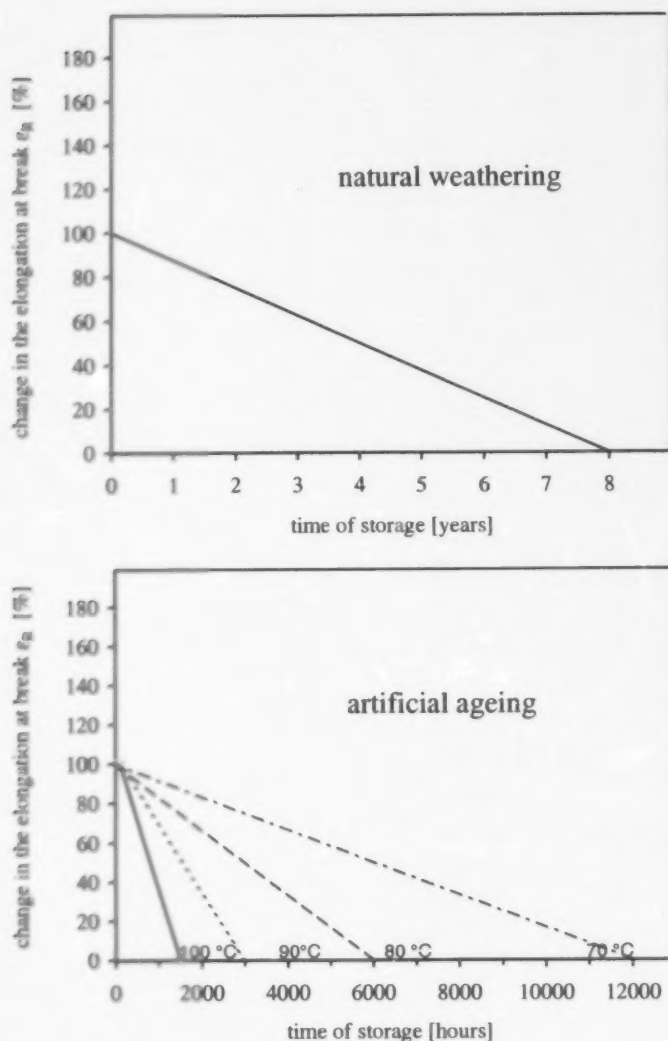


Fig. 1: Fictitious, ageing sensitive material, comparison of natural weathering and artificial ageing at various temperatures (idealised representation)

Table 2: Estimated time-compression factors at given temperatures

Test temperature °C	Test time, hrs	Time-compression factor
70	12.000	1: 6
80	6.000	1: 12
90	3.000	1: 24
100	1.500	1: 48
110	750	1: 96
120	375	1:192

3.1.2 Artificial versus natural ageing: a comparison

The difference between results obtained from open-air weathering and those from indoor storage have usually been slight for the five-year test periods used up to now. Ageing seems to be mainly a result of internal processes that take place within the 16 test materials involved. Certain materials showed that post-curing can take place over one year and actual ageing appears to only take place thereafter. The same post-curing was also determined in the initial period for the artificial ageing process described below.

Almost all the materials stood up well to artificial ageing at temperatures of between 60°C and 120°C. Correlation existed between artificial and natural ageing for all 16 elastomers tested. The frequency of cases of correlation was only slight reduced

by increasing the testing temperature during the temperature/time period used. For use in civil engineering, the upper testing temperature has previously shown as 70°C.

Non-ageing processes do not seem to exercise as much influence during longer test periods as they did over the shorter testing periods previously used. Changes of test environment, in most cases, have more effect than mere heat storage. The differences between the UVCON and SUNTEST series of tests are probably due less to differences in sensitivity to these radiation tests present in certain materials than to differences in average temperature. This is more apparent in the UVCON series of tests.

Based on the results obtained from the artificial ageing of free-film sealant products, the 16 materials evaluated in this study can be classified in essentially three different materials "types", corresponding to their response to artificial ageing and related physical characteristics. These are:

- i.) Type 1 materials have elongation at break (ϵ_R) that changes as a function of temperature in accordance with the Arrhenius equation whilst not having any evident nor significant effect on the strength at break. An example of this type of material is given in Figures 2-5 that shows percentage change in elongation at break in relation to ageing time and ageing temperatures. Type 1 materials include: PS9, PS10, PS22, PS 28, PU23, CR24 and E25.
- ii.) Materials characteristic of Type 2 suggest that the change in tensile strength at break (δ_R) follow the Arrhenius relationship whereas wlongation at break is not coupled to changes in temperature. An example of this type of material is

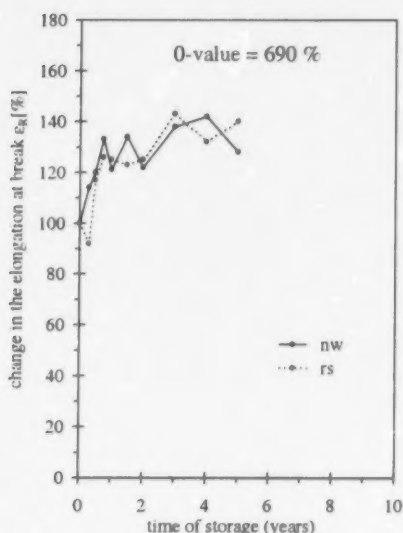


Fig. 2: Material PS 28 - natural weathering and room storage

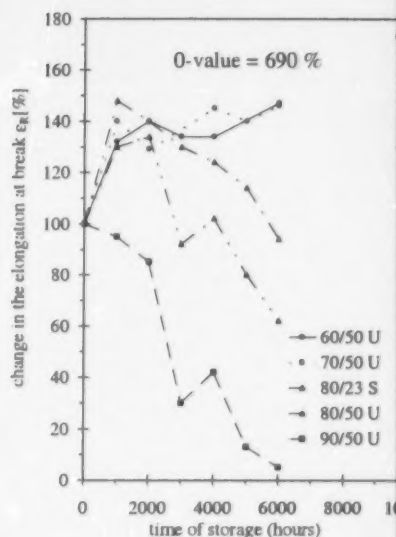


Fig. 3: Material PS 28 - accelerated ageing UVCON and SUNTEST

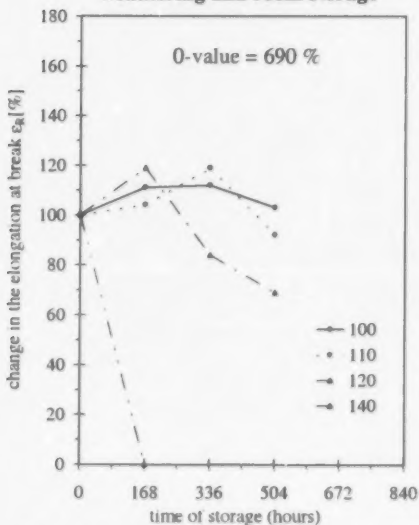


Fig. 4: Material PS 28 - accelerated ageing OVEN (100 ... 140 °C)

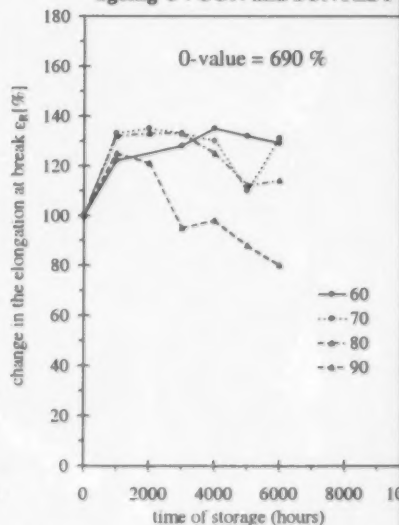


Fig. 5: Material PS 28 - accelerated ageing OVEN (60 ... 90 °C)

given in Figures 6-9. Materials in this category include products SI17 and PS12.

iii.) Type 3 materials are characterised by little or no aged-induced changes in a given ageing regime. Essentially, the materials are only slightly or completely insensitive to the effects of heat, UV and water in the exposure regimes and over the time intervals over which they were tested. This is evident in instances where the ageing effects act alone or indeed where the ageing period was prolonged from 6,000 hours to 12,000 hours. The data remains the same, or only change initially, since the material has not yet been fully cured. An example of this type of material is given in figures 10-13. Materials of this type include SI5, SI6, SI7, SI8, SI18, SI19 and SI21.

3.2 Verification trials on joint models

Ten of sixteen sealants tested were processed not only as a free film but also as a joint model. The ageing profiles of the change in tensile strength and elongation at break after 5 years of open air weathering were similar on all sealants to those of the free films. For the materials SI5, SI6, SI7, SI8 and PU23, it was possible to determine the full amount of tensile strength and elongation at break of the sealant, on the other materials, the measured data are too low, since adhesion breakage occur between the sealant and concrete.

The ageing profiles of the change in tensile strength and elongation at break were determined for materials SI5 and SI6 over 6,000hrs in the UVCON device with an alternating cycle of 8hrs UV at 80°C and 4hrs condensation at 50°C. These were similar to open air weathering.

On the joint model by comparison of the slimmer free film, postcuring occurs more slowly and the UV radiation has somewhat weaker effect.

3.3 Proposal for standard test methods

3.3.1 Free films or joint models

Free-film specimens should be used where adhesion of the substrate is not critical to evaluating the product. Specifically, this applies to those materials that are used as a jointing tape. Where previous trials have shown that the Sealant adhesion to substrate is likely to remain or indeed, increase over the long-term, then the use of free-film testing is considered adequate. In all other cases, tests on joint models are preferred.

3.3.2 Thermal or combined ageing

Thermal ageing alone is suitable for those elastomeric sealants that have only little, if any, sensitivity to UV radiation. Of the 16 materials evaluated in this study, the products that could be tested in this fashion include SI5, SI6, SI7, SI8, SI19, SI21, CR24 and E25. The simplicity of thermal ageing lends itself very well to accelerated ageing, e.g., it is possible to test these products at 120°C as opposed to the typical test temperature of 70°C. Testing at 120°C would provide a time compression-factor of 1:192 and this would permit estimating the effects of ageing over 11 years with a test conducted in 500 hours. The results could potentially be extrapolated to 22 years.

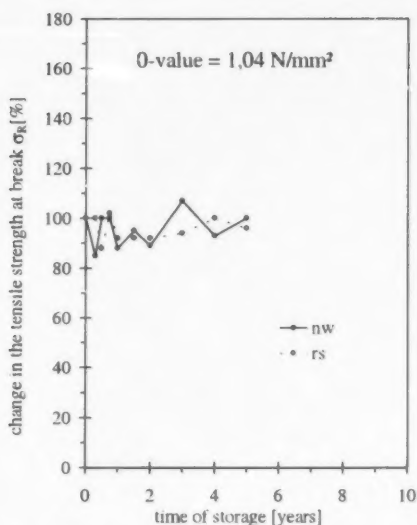


Fig. 6: Material SI 17 - natural weathering and room storage

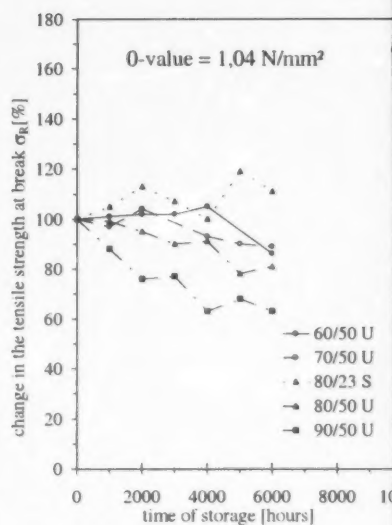


Fig. 7: Material SI 17 - accelerated ageing UVCON and SUNTEST

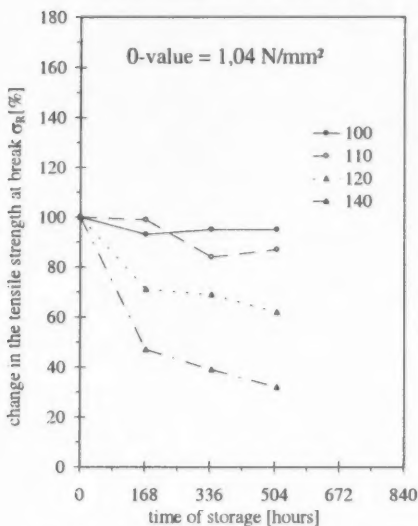


Fig. 8: Material SI 17 - accelerated ageing OVEN (100...140°C)

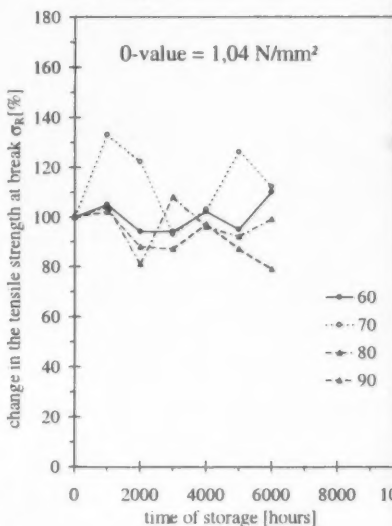


Fig. 9: Material SI 17 - accelerated ageing OVEN (60...90°C)

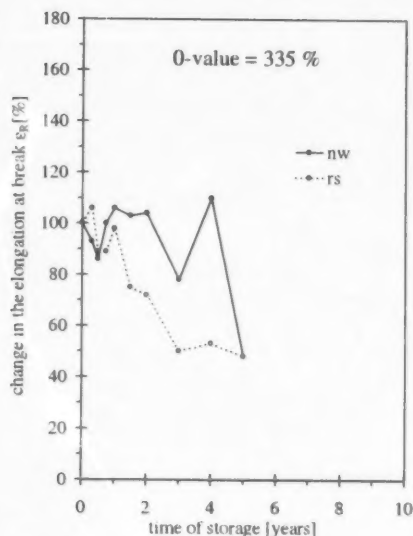


Fig. 10: Material SI 19 - natural weathering and room storage

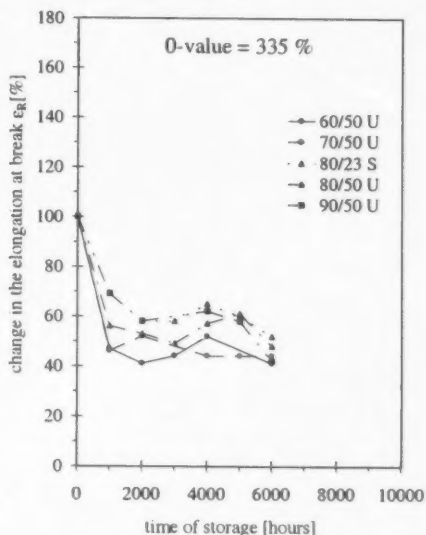


Fig. 11: Material SI 19 - accelerated ageing UVCON and SUNTEST

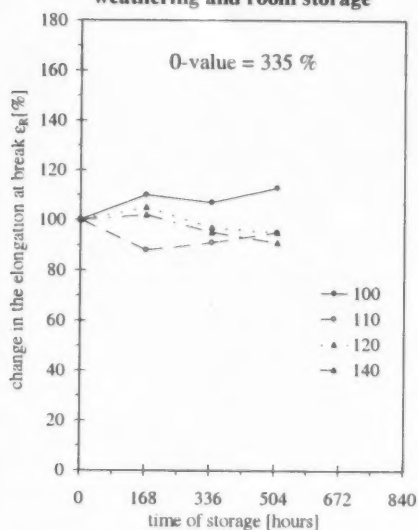


Fig. 12: Material SI 19 - accelerated ageing OVEN (100...140°C)

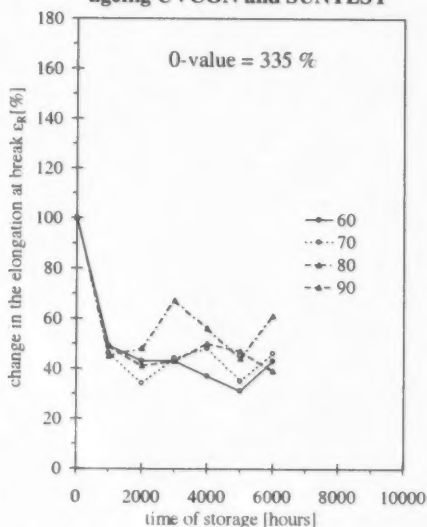


Fig. 13: Material SI 19 - accelerated ageing OVEN (60...90°C)

Combined ageing should be used for all products that have a certain higher degree of UV sensitivity. These materials include SI17, SI18, PS9, PS10, PS12, PS22, PS28 and PU23. The time-compression factor is 1:24 for tests conducted at 90°C and this implies a test time of 3.000 hours (18 weeks) that would simulate ageing over 8 years. Other possible suggestions include conducting the test over 500 hours (3weeks) such that 2,5 years of ageing is simulated. The temperature of 90°C mentioned above was determined by the operating limit of the UVCON weathering device used. The possibility of increasing the test temperature limit depends on the manufacturer of the test equipment. In the use of apparatus operating with Xenon arc radiation, for example, the expensive cooling system required to maintain a constant temperature could be replaced with a more valuable heating system, for the unit.

4 Conclusions affecting further work

The above report justifies, in the opinion of its authors, the use in the artificial ageing process of sealants of testing temperatures of up to 120°C. Cost and time savings are significant.

Further work is now required as a result of (presumable isolated) problems that emerged during testing. This concerns, for example, the non-ageing chemical reactions of certain sealants and the temperature-sensitivity of some substrates. Furthermore, the experimental classification of influencing factors such as movement and air pollution has not yet been concluded. The materials mentioned in section 2.1 are not yet sufficiently proven to permit their use as fully-calibrated test materials in future experiments.

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77 ON-SITE NON-DESTRUCTIVE TEST FOR SEALANTS

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Abstract

In recent years, the deterioration of sealants has attracted much attention in conjunction with the construction industry's increasing use of large panel curtain walls. Although sealants are relatively inexpensive, they constitute a major function in a building's life cycle. Effective sealants are important in contributing to successful watertightness and energy efficiency of the building envelope. The high cost of energy for heating and air-conditioning is causing serious consideration of good building sealants. The long term financial impact of poor sealants on the cost of keeping the building comfortable is now being calculated by building owners. Currently in the local scene, most sealants in high rise buildings are reaching the limit of their life span. Generally, sealants for cladding wall applications have a warranty period of 13 years or less. Reports have shown that many building in the city area are anticipated in the next few years, to engage in repair or replacement of sealants in the building facade. This paper discusses the development of an on-site non-destructive testing technique to assess the performance of sealants on building facades. Tests were carried out on various generic types of high performance sealants subjected to natural and artificial weathering. Results were compared with those from the conventional laboratory tests including elasticity, compression, adhesion and cohesion.

Keywords: elastic recovery, sealants, compressibility, test method, experiment, test device, correlation

1 Introduction

The evolution of modern architecture depicts an increasing trend of curtain walling construction. This trend is complemented by the usage of elastomeric sealants such as silicones, polysulphide and polyurethane. These polymers allow for more design creativity and are expected to accommodate mechanical vibration, thermal movement, expansion and contraction stemming from weather variances. The integrity of a building is thus maintained by the elastomeric properties of sealants that prevent water and air infiltration, thereby warranting a watertight and airtight envelope.

In the case of determining the durability and performance of new sealants, the conventional testing techniques stipulates that the sealants must be statically cured for a period of 28 days before the tests are to be conducted. This approach seems to suggest that all sealants fail from weathering, ignoring the fact that failure may actually occur before the sealant has a chance to cure. In practice, sealants that are installed on-site, do not "enjoy" the luxury of being statically cured in a fixed position, before experiencing undue stress caused by movements. They are subjected to dynamic curing rather than static curing and occurrences of premature failures are highly possible. This highlights the concern that conventional control tests conducted may be meaningless if sealants were to fail before they are put to service. It is thus important to monitor the in-service performance of sealants regularly especially during the initial stage after application (Margeson, 1992; Matsumoto, 1992; Beech & Beasley, 1992; Klosowski, 1978; Panek and Cook, 1984). To achieve that, a reliable on-site, non-destructive diagnostic technique is needed.

This paper gives an overview of a research project looking into the feasibility of developing an in-situ non-destructive diagnostic technique for the assessment of sealants based on elastic recovery and compressibility.

2 Methodology

The project was broadly divided into three main sections with their respective experiments as follows:

Section I Development of Conceptual framework

- | | |
|----------------|---|
| Experiment 1 - | Semi-destructive elastic recovery test method versus ASTM D412 test method. |
| Experiment 2 - | In-situ non-destructive elastic recovery test method versus standard resilience test. |
| Experiment 3 - | In-situ non-destructive elastic recovery test method - horizontal and vertical orientated |
| Experiment 4 - | Semi-destructive elastic recovery test method versus in-situ non-destructive elastic recovery test method |
| Experiment 5 - | Investigation into the in-situ compressibility test |
| Experiment 6 - | Investigation into the correlation between compressibility with elastic recovery and Shore A hardness |

Section II Assessment of the reproducibility, repeatability, sensitivity and reliability on in-situ non-destructive elastic recovery test device

- Experiment 7 - The significance of the shape of the detector
- Experiment 8 - The significance of the size of the detector
- Experiment 9 - The significance of the restoring time
- Experiment 10 - The significance of the type of substrate
- Experiment 11 - The significance of temperature
- Experiment 12 - The significance of moisture content
- Experiment 13 - The significance of air-void
- Experiment 14 - The reproducibility of the in-situ non-destructive test device
- Experiment 15 - The repeatability of the in-situ non-destructive test device

Section III Applicability of in-situ non-destructive elastic recovery test device

- Experiment 16 - The effect of curing on the elastic recovery of sealant
- Experiment 17 - The applicability of compressibility test
- Experiment 18 - Adhesion and cohesion of sealant

Five generic types of high performance sealants were evaluated which were chosen to represent the wide range of materials available in the market. These were one part silicone (1S), one part polysulphide (1P), one part polyurethane (1U), two-part polysulphide (2P) and two part polyurethane (2U). Experiments were conducted on the sealants that were exposed to various simulated weathering conditions such as water immersion at 25°C and 70°C, heat aging at 70°C and QUV weatherometer. Three main types of sealant specimens were fabricated i.e.:



Fig. 1: Aluminium mould of dimension 30mm (length) x 15mm (height) x 10 mm (width) that resembles the "as-built" sealants used in joints of buildings



Fig. 2: H-joint configuration sealant specimens with different types of substrates i.e. glass, mortar and aluminium prepared similar to that of ASTM C719-93



Fig. 3: Dumbbell shape according to ASTM D412

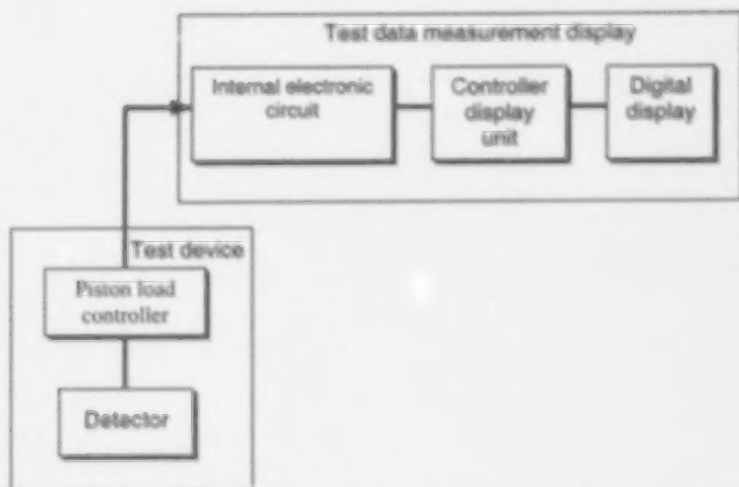


Fig. 4: A schematic diagram of the in-situ non-destructive test device

3 Results and Discussion

The results are broadly summarised in Tables 1 and 2 in the following pages.

In the first section, a semi-destructive elastic recovery test device was developed based on the principle of ASTM D412 test method. Sealants were cast in aluminium moulds to simulate the actual sealant joints on-site. This semi-destructive elastic recovery test method involved the modification of a standard laboratory test ASTM D412 for elastic recovery of sealant into an on-site diagnostic technique. The evaluation and correlation of the existing laboratory testing technique (ASTM D412) with the semi-destructive elastic recovery test method were carried out. Good correlation was achieved. The project extended on to the development of an in-situ non-destructive elastic recovery test device (Figure 4). Special attention is dedicated to the simulation of real life situation such as the shapes and dimensions of the mould and the plane of testing when conducting the experiments. Results derived from the test device were correlated with the standard resilience test both vertically and horizontally as well as with the semi-destructive test method. Good correlation was achieved in various aspects. Further enhancements and improvement such as the inclusion of testing facilities for compressibility and visual assessment were incorporated. The compressibility test has also been correlated with the elastic recovery and Shore A hardness.

In the second section of the development work, laboratory experimental studies were carried out to assess and evaluate the significance of various factors that might affect the reliability and sensitivity of the test device. Among the factors investigated were the shape of detector, size of the detector, types of substrates, restoring time, temperature variation, moisture content variation and the compactness/air-void. The reproducibility and repeatability of the test device were also investigated.

The viability of the test device was evaluated using statistical analysis on data derived from in-situ non-destructive elastic recovery tests in which the degree of correlation among different sets of experiments was determined. In addition, attempts were made to establish the relationship between the various tests on high performance sealants.

The third section looked into the application aspects such as the effect of the curing process. Curing is very crucial to the development of strength, movement capability and elastic recovery material. Inferences from the experiment will thus serve as a primary indication and guideline to be utilised as a benchmark for the industry. It is apparent from the results that silicones and two part sealants are superior in term of cure rate, as compared to one part polysulphide and polyurethane, engendering them to be more desirable for sealing joints which are subjected to high movement stress immediately upon installation.

The compressibility test used to assess hardness and detect voids in sealants, in terms of applied force, was also investigated. The test device has shown to be reliable in detecting voids and measuring hardness in terms of applied force. The compressibility test has been shown to be more superior than Shore A hardness test device in terms of sensitivity and consistency.

Table 1: Development of in-situ non-destructive elastic recovery test device

Test methods			Plane of orientation test		Sealant	Linear relationship equation	R ²
Y	X		Vertical	Horizontal			
ASTM D412	Semi-destructive		-	-	simulated	$y = 0.1116x + 85.346$	0.8237
Non-destructive	Resilience test		-	Horizontal	Fresh	$y = 0.9847x - 6.2599$	0.9665
Non-destructive	Resilience test		-	Horizontal	Fresh	$y = 0.8735x - 3.182$	0.9343
Non-destructive	Non-destructive		Vertical	Horizontal	Fresh	$y = 0.8842x + 2.3878$	0.9646
Non-destructive	Non-destructive		Vertical	Horizontal	simulated	$y = 1.0858x - 10.229$	0.9702
Non-destructive	Semi-destructive		Vertical	-	simulated	$y = 0.6159x + 33.565$	0.8375
Elastic recovery	Compressibility		Vertical	-	Fresh	$y = -0.0414x + 94.485$	0.0013
Elastic recovery	Compressibility		Vertical	-	simulated	$y = 0.0245x + 76.07$	0.0003
Shore A hardness	Compressibility		Vertical	-	Fresh	$y = 1.1175x + 14.53$	0.5764
Shore A hardness	Compressibility		Vertical	-	simulated	$y = 0.606x - 7.7815$	0.8257

Table 2: Evaluation and correlation of linear relationship equations on the assessment of the reliability of the in-situ non-destructive elastic recovery test device

Assessment	Factors		Plane of orientation test		Sealant	Linear relationship equation	R ²
	(X)	(Y)					
Shape of detector	Pellet	Ball	Vertical	Horizontal	Simulated	$y = 0.8865x + 10.059$	0.8732
	Pellet	Ball	Vertical	-	Simulated	$y = 0.8839x + 12.53$	0.86
	Pellet	Ball	-	Horizontal	Simulated	$y = 0.8972x + 6.8472$	0.5847
Shape of detector (Sensitivity)	Pellet	Ball	Vertical	Horizontal	Simulated	$y = 0.8958x + 2.9416$	0.8695
	Pellet	Ball	Vertical	-	Simulated	$y = 0.8765x + 4.136$	0.8306
	Pellet	Ball	-	Horizontal	Simulated	$y = 0.9205x + 1.3972$	0.9027
	Pellet	Ball	Vertical	Horizontal	Simulated	$y = 0.8189x + 17.294$	0.9102
Size of detector	10mm pellet	5 m pellet	Vertical	Horizontal	Simulated	$y = 0.7836x + 20.8$	0.9006
	10mm pellet	5 m pellet	Vertical	-	Simulated	$y = 0.8533x + 13.897$	0.9239
	10mm pellet	5 m pellet	-	Horizontal	Simulated	$y = 0.8089x - 1.1077$	0.8892
Size of detector (Sensitivity)	10mm pellet	5 m pellet	Vertical	Horizontal	Simulated	$y = 0.7562x + 1.9314$	0.887
	10mm pellet	5 m pellet	-	Horizontal	Simulated	$y = 0.8724x - 0.3424$	0.9157
	20 s	5 s	Vertical	Horizontal	Simulated	$y = 0.9503x - 0.0703$	0.928
	40 s	5 s	Vertical	Horizontal	Simulated	$y = 1.0113x - 7.0493$	0.9214
Restoring time	60 s	5 s	Vertical	Horizontal	Simulated	$y = 0.9859x - 7.7902$	0.8986
	20 s	5 s	Vertical	Horizontal	Simulated	$y = 0.8951x + 2.103$	0.8711
	40 s	5 s	Vertical	Horizontal	Simulated	$y = 0.9405x + 3.7278$	0.8654
	60 s	5 s	Vertical	Horizontal	Simulated	$y = 0.8834x + 4.1333$	0.812
Substrate types	aluminium	glass	Vertical	-	Curing	$y = 0.9745x + 1.4942$	0.9829
	mortar	aluminium	Vertical	-	Curing	$y = 0.9792x + 2.3686$	0.9607
	mortar	glass	Vertical	-	Curing	$y = 0.9625x + 3.2764$	0.9595

Table 2, cont'd: Evaluation and correlation of linear relationship equations on the assessment of the reliability of the in-situ non-destructive elastic recovery test device

Assessment	Factors	Plane of orientation test	Sealant	Linear relationship equation	R ²
Temperature of sealant	30°C - 70°C	Vertical	Fresh AS101(1U)	$y = 0.0512x + 89.579$	0.063
	30°C - 70°C	Vertical	Fresh SIU (1S)	$y = -0.0466x + 101.13$	0.2056
	30°C - 70°C	Vertical	Fresh T600(2P)	$y = -0.0502x + 100.91$	0.0977
	30°C - 70°C	Vertical	Fresh T1 (1P)	$y = 0.348x + 46.061$	0.285
Moisture content of sealant	Variation in moisture content (%)	Vertical	Fresh PE (1P)	$y = 24.596x + 64.34$	0.7033
		Vertical	Fresh AS101(1U)	$y = 0.2447 + 94.796$	0.0046
		Vertical	Fresh NI525(1U)	$y = 3.1019x + 84.838$	0.3393
		Vertical	Fresh BF (1U)	$y = 0.7294x + 68.897$	0.1938
		Vertical	Fresh BBS (1S)	$y = -0.0711x + 100.02$	0.0042
		Vertical	Fresh T600(2P)	$y = -0.3973x + 96.73$	0.0175
Air-void	Full compactness	air-void	Simulated	$y = 1.021x - 0.6694$	0.9708
Reproducibility	Device 1	Device 2	Curing	$y = 1.1504x - 9.7828$	0.9269
Repeatability	Testing 1	Testing 2	Simulated	$y = 0.9727x - 1.7654$	0.8598

The failure of building sealants in an active joint is usually manifested by cohesive failure in the sealant or adhesive failure between the sealant and substrates or both. Thus, another aim of the applicability studies was to examine and correlate the three performance parameters of high performance sealant that is elastic recovery, cohesion and adhesion when subjected to accelerated weathering. Laboratory studies based on cyclic movement (Hockman Cycle ASTM C719-93) were conducted. In the cyclic movement studies, mortar substrates H-joint configuration sealant specimens were fabricated and subjected to cyclical simulations up to 5 cycles. The conditions consisted of water immersion at 25°C followed by oven treatment at 70°C and 10 cycles of alternate compression and extension joint movement in a compression-extension cyclic machine. Upon completion of the cyclical simulation, elastic recovery was measured and the magnitude of the cohesive and adhesive failures were determined and evaluated. Analysis was rendered to examine the relationship of elastic recovery performance with cohesive and adhesive failures of the various generic sealants. The relationship was traced through five cycles of laboratory testing and subsequently correlation was drawn to affirm the reciprocity among the parameters. A correlation was established to evaluate the relationship between the three parameters, i.e. cohesion, adhesion and elastic recovery.

4 Conclusions

In the development framework of the in-situ non-destructive elastic recovery test device, correlation of results derived using different elastic recovery test methods i.e. ASTM D412, semi-destructive elastic recovery test method, in-situ non-destructive elastic recovery test method, standard resilience test and compressibility test were established. A wide range of different generic types of sealants was tested to ensure that the works might be applicable to a broad range of sealants subjected to different degree of weathering.

The test device was subsequently evaluated for its reliability, sensitivity and the extent to which it would be influenced by factors such as shape of detector, size of detector, restoring time, types of substrates, temperature variation, moisture content variation, compactness and air-void.

The significance of curing on the performance of sealants has been investigated. The in-situ non-destructive elastic recovery test device has been shown to be useful on the monitoring of the physical curing process of various generic sealants. Works are still being undertaken to determine the long term curing characteristics of the sealants to establish the benchmark that can be used when assessing the state of sealants on-site.

Compressibility test has been shown to be a parameter for determining air-void/compactness. The test has been shown to be more superior than Shore A hardness test device in term of sensitivity and consistency.

In the cyclical movement studies, the effect and extent of both the cohesive and adhesive failures on the performance of elastic recovery of sealants were examined. The results have shown no correlation between adhesive failure and elastic recovery. However, good correlation was found between cohesive failure and elastic recovery of the sealant. Critical elastic recovery of the sealant can be determined through the study of cyclical movement. Melting and softening may also cause low elastic recovery regardless of the extent of the cohesive failure.

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78 NEW PREDICTION METHODS OF SERVICE LIFE OF POLYMERIC FINISHING MATERIALS

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Abstract

Polymeric surface finishing materials not only exhibit such functions as beauty and water proofness, but also play the role of protective function against the deterioration of structural members by shielding a substance from external deterioration factors such as carbon dioxide and sea salt particles. They are liable to deteriorate as the change of materials properties, however, under outdoors exposed condition, mainly by the action of ultraviolet light as well as by other supplemental factors such as oxygen, water, and heat. In ordinary prediction methods of service life of these polymeric finishing materials, deterioration models were considered by the change of material properties with time as the multiple processes of photo-, thermal-, and hygroscopic- deterioration by the simultaneous action of the constant deterioration factors of ultraviolet, heat and moisture.

Depletion of ozone layer as one of the global environmental problems are becoming more and more appreciated to increase the dangerous shorter wavelength ultraviolet light, and is expected to influence the deterioration mechanisms, and accelerate the deterioration of polymeric finishing materials, resulting in making their service life much shorter. In this paper, by considering this tendency of the increase in the dangerous shorter wavelength ultraviolet light, rational deterioration models for new prediction methods of service life of polymeric finishing materials are proposed, based upon exponential monotone increasing function with time of photon flux activating the polymer molecules. It is assumed that the deterioration of polymers occurs by the two fundamental processes of photochemical depolymerization and photooxidation. Computer simulations are done for these processes. Accelerated weatherability test methods based upon a step function modeling this tendency are also reported. The comparison of these two cases with the case of constant photon flux are reported.

Keywords: acceleration, computer simulation, deterioration, depletion of ozone layer, photon flux, polymeric finishing materials, prediction of service life, ultraviolet light

1 Introduction

Building polymeric finishing materials not only show functions of the beauty and water proofness etc., but also play the important role of the protection against deterioration of such structural members as steel reinforced concrete by shielding the steel reinforcement against external deterioration factors such as atmospheric carbon dioxide (Fukushima 1994). Under outdoor exposure, however, they are liable to deteriorate as the change of material properties by the action of ultraviolet light and other factors. There are many deterioration factors such as ultraviolet light, oxygen, water and heat etc., but ultraviolet light is the main deterioration factor.

When the deterioration of polymeric materials is discussed, prediction methods of the progress of the change of material properties based upon models considering primarily the action of ultraviolet light, and taking into account the subsidiary effects of oxygen, water and heat are general in various fields (Kashiwagi et al. 1985; Martin 1984). Then simplification by considering the deterioration factors as constant, not depending on time is done. That is also the case in building field.

Fukushima (1986) has already set the theoretical model for the deterioration processes of building polymeric finishing materials based upon the simultaneous photochemical action of activated polymeric molecules by absorbing ultraviolet light and diffusive oxygen from the surface, and carried on unsteady state dynamic analysis, deriving the parabolic law concerning time for the increase of the depth of deterioration layer. Further Fukushima and Motohashi (1986) showed that their deterioration is reflected as the reduction of suppressive function against the progress of carbonation of concrete.

Depletion of ozone layer as one of the important global environmental problems, however, causes the increase of harmful shorter - wavelength ultraviolet rays (UV-B light (285~315^{nm}))(UNEP 1994). It is expected that these ultraviolet rays have influence on mechanisms of deterioration of building polymeric finishing materials, accelerating the progress of their deterioration. We should establish new prediction methods of deterioration and counter-measures in advance, corresponding to these situations in the coming twenty-first century.

This paper deals with such new prediction methods of service life of polymeric finishing materials (Fukushima 1995). Considering the tendency of the increase in the dangerous shorter wavelength ultraviolet light, rational deterioration models for new prediction methods of service life of polymeric finishing materials are proposed, based upon exponential monotone increasing function with time of photon flux activating the polymer molecules. It is assumed that the deterioration of polymers occurs by the two fundamental processes of photochemical depolymerization and photooxidation.

Further modeling this tendency of exponential increase of photon flux by a step function, the author tries to establish a new accelerated weatherability test method by step response method (Fukushima 1996).

2 Research methods and research results

2.1 Unsteady state dynamic analysis of deterioration processes

2.1.1 Theoretical model

The author carries on unsteady state dynamic analysis for deterioration processes of organic polymeric materials based upon photochemical reactions concerning activated polymer and diffusive oxygen. It is assumed that the shift effect of optical spectra of ultraviolet light towards shorter wavelength is reflected as the exponential increase with time of the amount of ultraviolet rays near the characteristic absorptive band causing the deterioration (Fig. 1).

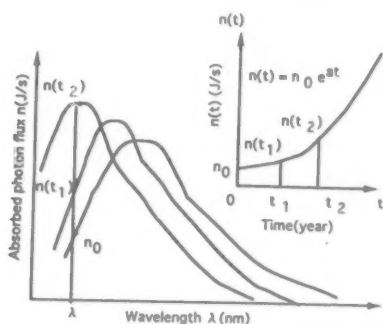


Fig. 1: Concept of increase with time of ultraviolet photon flux absorbed near characteristic absorption band causing the deterioration of polymeric molecules

The fundamental processes of theoretical model for the unsteady state dynamic analysis are as follows:

- 1) Only activated polymeric molecules by absorbing ultraviolet rays near the characteristic absorptive band (wavelength; λ , photon flux; $n(t) = n_0 e^{at}$, n_0 : initial photon flux, a : increase coefficient of ultraviolet photon flux, t : exposure time) contribute to photochemical reactions (efficiency; η , molar concentration C^*), polymeric molecules in ground state not contributing.
- 2) Photochemical reactions are due to the following two ones :
 - i) Depolymerization of polymeric molecules (first-order reaction: $R_1 = k_1 C^*$)
 - ii) Photooxidation by the collision of diffusive oxygen (C_{AO} , C_A : surface and internal molar concentration, D_A : diffusion coefficient) with activated polymeric molecules (second order reaction: $R_2 = k_2 C^* C_A$. This reaction can be considered to be pseudo-first-order one ($R_2 = k'_1 C_A$), considering that the diffusion of oxygen becomes the rate-ruling stage.

- 3) Deteriorated polymeric molecules (molar concentration ; C_B) remain at the original site, not diffuse.
- 4) Influence of temperature on the photochemical reactions and diffusion are incorporated in to rate constants (k_1 , k_2) and effective diffusion coefficient (D_A), by obeying the Arrhenius' law.
- 5) Deterioration progresses from the surface to inner layers, and the degree of deterioration changes with time (t) and the depth from the surface (x). As the deterioration progresses, such infrared (IR)-active functional groups as carbonyl ($-CO$) are produced, and the degree of deterioration is reflected as the increase in absorptive strength (α'_x) in the characteristic IR absorption wavelength (λ'). Deterioration layers do not contribute to functions of polymeric finishes and course the reeducation of function, for example, the increase in air permeability.
- 6) Compared with the total depth of polymeric finish, that of deteriorated layer is smaller enough for us to consider the total depth as virtually indefinite by large.

2.1.2 Fundamental equations of analysis

- 1) In case of considering the exponential monotone increase of the amount of ultraviolet light with time.

Fundamental equations for theoretical analysis with initial and boundary conditions are summarized as follows:

$$\partial C_A / \partial t = D_A \partial^2 C_A / \partial x^2 - k_1' C_A \quad (1)$$

$$\partial C^* / \partial t = (\eta n_0 e^{at} / N_0) C_0 \epsilon_\lambda \exp(-C_0 \epsilon_\lambda X) - k C^* - k_1' C_A \quad (2)$$

$$\partial C_B / \partial t = k_1 C^* + k_1' C_A = b (\partial \alpha'_x / \partial t) \quad (3)$$

$$[\text{Initial Condition}]; \quad t \leq 0 : C^* = C_A = C_B = 0$$

$$[\text{Boundary Conditions}]; \quad \text{[I]} t > 0, x = 0 : C_A = C_{AC}$$

$$\text{[II]} t > 0, x \rightarrow \infty : C_A = 0$$

$$\text{[III]} t > 0, x = \delta : \partial C_B / \partial x = 0$$

In which : N_0 ; Avogadro number, C_0 ; initial molar concentration of undeteriorated polymer molecules, ϵ_λ ; molar absorption coefficient, b ; proportion coefficient

- 2) In case of setting the amount of ultraviolet light as constant, not depending on time. In this case, equation (2) is replaced by the equation (2)'.

$$\partial C^* / \partial t = (\eta n_0 / N_0) C_0 \epsilon_\lambda \exp(-C_0 \epsilon_\lambda X) - k C^* - k_1' C_A \quad (2)'$$

2.1.3 Analytical results

$$C_A(x, t) = \frac{C_{AO}}{2} \left\{ \exp \left(-x \sqrt{\frac{k_1}{D_A}} \right) \operatorname{erfc} \left(\frac{x}{2\sqrt{D_{AO}}} \cdot \sqrt{k_1 t} \right) + \exp \left(x \sqrt{\frac{k_1}{D_A}} \right) \operatorname{erfc} \left(\frac{x}{2\sqrt{D_A t}} + \sqrt{k_1 t} \right) \right\} \quad (4)$$

$$C^o(x, t) = C_o \exp(-C_o \varepsilon_A x) \frac{e^{at} - e^{-k_1 t}}{a + k_1} - k_1 C_{AO} e^{-k_1 t} \int_0^t \varphi_A(x, \tau) e^{-k_1 \tau} d\tau \quad (5)$$

$$C^s(x, t) = C_o \exp(-C_o \varepsilon_A x) \frac{1 - e^{-k_1 t}}{k_1} - k_1 C_{AO} e^{-k_1 t} \int_0^t \varphi_A(x, \tau) e^{-k_1 \tau} d\tau \quad (5)^*$$

$$C_B(x, t) = C_o \exp(-C_o \varepsilon_A x) \frac{k_1 e^{at} - a e^{-k_1 t}}{a(a + k_1)} - k_1 k_1 C_{AO} e^{-k_1 t} \int_0^t e^{-k_1 u} du \int_0^u \varphi_A(x, \tau) e^{-k_1 \tau} d\tau + k_1 C_{AO} \varphi_A(x, t) dt \quad (6)$$

$$C_B(x, t) = \frac{C_o \exp(-C_o \varepsilon_A x)}{k_1} \{k_1 t - [1 - \exp(-k_1 t)]\} - k_1 k_1 C_{AO} \int_0^t e^{-k_1 u} du \int_0^u \varphi_A(x, \tau) e^{-k_1 \tau} d\tau + k_1 C_{AO} \int_0^t \varphi_A(x, \tau) d\tau \quad (6)^*$$

2.2 Application for new accelerated weatherability test - step response method

When we try to take the tendency of exponential increase with time of ultraviolet photon flux into accelerated weatherability test, sophisticatedly-controlled ultraviolet sources are needed, which are technically rather difficult, considering the present situations. Consequently, by modeling such tendency by a Gaussian-type step function (Fig. 2), the author tries to establishing a new accelerated weatherability test method by step response analysis.

In this case, the eqn.(2) is replaced by the eqn.(2)'.

$$\frac{\partial C^*}{\partial t} = \eta \eta_o \left\{ \operatorname{floor} \left(\frac{t}{T} \right) + 1 \right\} C_o \varepsilon_A \exp(-C_o \varepsilon_A x) - k_1 C^* - k_1 C_A \quad (2)''$$

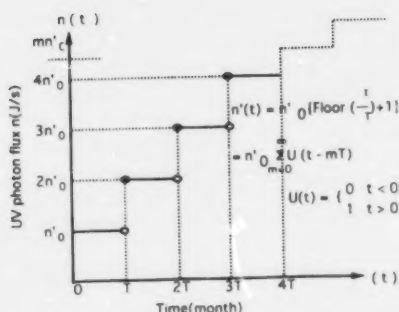


Fig. 2: Gaussian Step fraction of ultraviolet photon flux

By solving eqns. (1), (2)', (3) also by using Laplace Transformation method, eqns (4), (5)', (6)' are obtained as analytical solutions.

$$C_A(x, t) = \frac{C_{AO}}{2} \left\{ \exp \left(-x \sqrt{\frac{k_i}{D_A}} \right) \operatorname{erfc} \left(\frac{x}{2\sqrt{D_{AO}}} - \sqrt{k_i t} \right) + \exp \left(x \sqrt{\frac{k_i}{D_A}} \right) \operatorname{erfc} \left(\frac{x}{2\sqrt{D_{AO}}} + \sqrt{k_i t} \right) \right\} \quad (4)$$

$$C^*(x, t) = C_o \exp(-C_o E_A x) \sum_{m=0}^{\infty} \frac{1 - e^{-k_i(t+mT)}}{k_i} - k_i C_{AO} e^{k_i t} \int_0^t \varphi_A(x, t) e^{k_i t} dt \quad (5)''$$

$$C_B(x, t) = C_o \exp(-C_o E_A x) \sum_{m=0}^{\infty} \left\{ t + \frac{1 - e^{-k_i t}}{k_i} e^{-mk_i T} \right\} - k_i k_i C_{AO} \int_0^t e^{k_i u} du \int_0^u \varphi_A(x, t) e^{k_i t} dt + k_i C_{AO} \int_0^t \varphi_A(x, t) \quad (6)''$$

2.3 Computer simulation

The difference among three cases of constant, exponential and step functional increase of ultraviolet photon flux is reflected in the first terms in the eqns. (6), (6)', (6)''. The profiles of the first terms of the concentration functions of deteriorated polymer molecules are shown in Figs. 3-8 by calculating by a personal computer (power Macintosh : Model Q650) and the mathematical processing software(Mathematica Ver.3.0).

```

 $\psi_1(x, t) = \{\exp(-C_0 a_\lambda x) /$ 
 $k_1\}$ 
 $\times \{k_1 t - [1 - \exp(-k_1 t)]\}$ 
 $k_1 = 0.1 [1/\text{year}] \quad C_0 a_\lambda = 1.0$ 
Plot3D[Exp[-x] (t-120.+12
0.Exp
[-0.0083t]), {x, 0, 10}, {t,
0, 800},
PlotRange->{0, 800}]

```

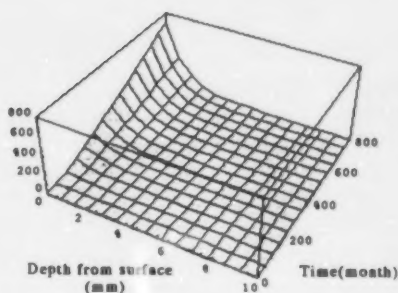


Fig. 3: 3D plot of the first term of concentration function of deteriorated polymeric molecule (in case of constant UV photon flux)

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 $\psi_2(x, t) = \exp(-C_0 a_\lambda x) \times \{k_1 \exp(a t + a e - k_1$ 
 $t) / \{a(a + k_1)\}\}$ 
 $k_1 = 0.5 [1/\text{year}], \quad a = 0.1 [1/\text{year}]$ 
 $C_0 a_\lambda = 1.0$ 
Plot3D[Exp[-x] (0.1Exp[0.1t]+0.1Exp
[-0t])/(0.1+0.2), {x, 0, 10}, {t, 0, 100},
PlotRange->{0, 150000}]

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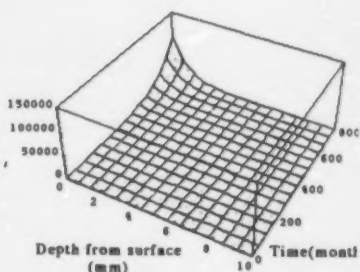


Fig. 4: 3D plot of the first term of concentration function of deterioration polymeric molecule (in case of exponential functional UV photon flux)

```

 $\psi_3(x, t) = \exp(-C_0 a_\lambda x) \times \sum_{m=0}^{\infty} [t +$ 
 $\{(1 - \exp(-k_1 t)) \exp(-m k_1 t) / k_1\}]$ 
 $m = 4 \quad k_1 = 0.1$ 
Plot3D[Exp[-x] (Sum[t+10.-10.Exp
[-(0.1t)i]-10.Exp[-0.1t]Exp[-
(0.1t)i], {i, 0, 4.})],
{x, 0, 10}, {t, 0, 800}, PlotRange->
{0, 5000}]

```

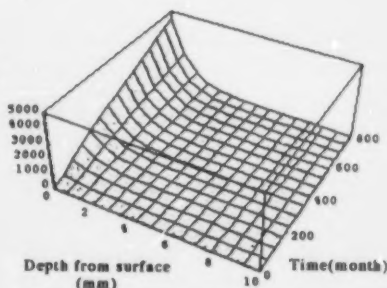


Fig. 5: 3D plot of the first term of concentration function of deteriorated polymeric molecule (in case of step functional UV photon flux)

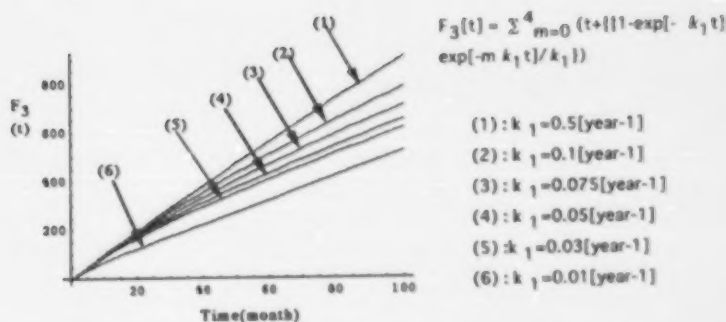


Fig. 6: Profiles of the first term of concentration function of deteriorated polymeric molecule (in case of constant function UV photon flux)(parameter; k_1)

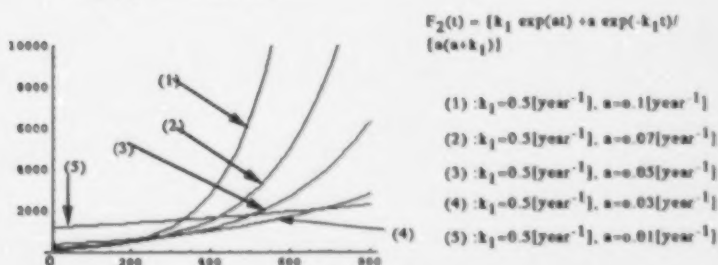


Fig. 7: Profiles of the first term of concentration function of deterioration polymeric molecule (in case of exponential functional UV photon flux)(parameter; a)

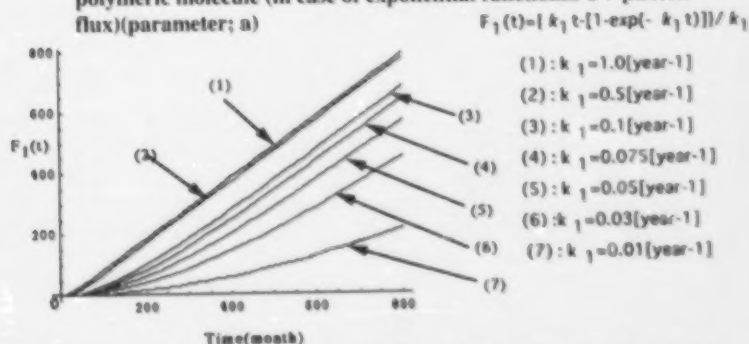


Fig. 8: Profiles of the first term of concentration function of deterioration polymeric molecule (in case of step functional UV photon flux)(parameter; k_1)

2.4 Change with time of the depth of deteriorated layer

The change with time of depth of deteriorated layer is obtained by the following relationship:

$$t > 0, x = \delta; \frac{\partial C_B}{\partial x} = 0 \quad (8)$$

Here the following mathematical formula is used.

$$\frac{\partial}{\partial x} \left(\int_0^t \Phi_A(x, t) dt \right) \Big|_{x=\delta} = \int_0^t \frac{\partial}{\partial x} (\Phi_A(x, t)) \Big|_{x=\delta} dt \quad (9)$$

By using appropriate approximations, the change with time(t) of the depth(δ) of deteriorated layer is summarized as follows:

- a) In case of constant ultraviolet photon flux:
(The following parabolic law is derived.)

$$\delta = a\sqrt{t} \quad (10)$$

- b) In case of exponential functional increase of photon flux:
(The following power law is derived.)

$$\delta = a\sqrt{t} + \frac{f(a)}{\sqrt{t}} = \beta t^n \quad (11)$$

- c) In case of step functional increase of photon flux:
(The following modified parabolic law is derived.)

$$\delta = a[1 + g(T)]\sqrt{t} \quad (12)$$

3 Discussion

From Fig. 3 to 8, we can see the situations of the first terms of concentration functions of deteriorated polymer molecules as follows:

In case of constant ultraviolet photon flux, the first term shows the linear increase with time, when rare constant of polymerization (k_1) are very near 1.0. In proportion as:

- 1) k_1 becomes smaller, however, it becomes exponential. The change with time of the depth of deteriorated layer obeys almost the parabolic law.
- 2) In case of exponential functional increase of ultraviolet photon flux, the first term shows the drastic exponential increase with time, and the degree of increase is controlled by increase coefficient of ultraviolet photon flux (a). The change with time of the depth of deteriorated layer obeys almost the power law

- 3) In case of step functional increase of ultraviolet photon flux, the first term shows almost the linear increase with time, when k_1 are very small. In proportion as k , becomes larger, it becomes almost the parabolic law having the convex curve upwards, going through an origin. The degree of increase in this case becomes much larger than in case of constant ultraviolet photon flux. The change with time of the depth of deteriorated layer obeys the modified parabolic law influenced by the step time interval.

4 Conclusion

Considering the tendency of the increase in the dangerous shorter wavelength ultraviolet light by the depletion of ozone layer, rational deterioration models for new prediction methods of service life of polymeric finishing materials are proposed, based upon exponential monotone increasing function with time of photon flux activating the polymer molecules. As the results, the following concluding remarks are obtained.

- 1) The influence of the tendency of increase with time of ultraviolet photon flux is reflected as the first term expressing the time dependence of the concentration function of deteriorated polymer molecules, and is controlled by increase coefficient of ultraviolet photon flux (a).
- 2) In proportion as the ultraviolet photon flux on creases, the time dependence of the depth of deteriorated layer becomes from the parabolic law to the power law.
- 3) Step response method by a Gaussian-type step function modeling the tendency of exponential increase with time of ultraviolet photon flux is effective enough to be a new accelerated weatherability test corresponding to the by the depletion of ozone layer in the coming twenty-first century.

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79 CERAMIC TILE FINISHING SYSTEMS FOR EXTERIOR USING ORGANIC ADHESIVES

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Abstract

In Japan, the prevention of delamination of ceramic tiling on exterior walls has been an important research theme in the building finishing technology field and has received extensive study. One method for preventing delamination is the application of a newly developed organic elastic adhesive. The representative material consists of modified silicone and epoxy resins. This material has a large capacity for deformation after curing and is used for ceramic tile finishing and stone finishing onto the concrete surfaces of exterior walls. In this study, the organic adhesives and adhesion systems were experimentally estimated. As a result of this study, quality criterion for the material is proposed. Also proposed are performance evaluation guidelines for exterior tile finishing panels and stone finishing precast concrete panels using the organic adhesives. In addition, standard construction specifications for exterior ceramic tile finishing and stone finishing systems with organic adhesives are proposed.

Keywords: Adhesion system, ceramic tile finishing, construction standard specifications, exterior wall, organic elastic adhesives, stone finishing

1 Introduction

In Japan, the prevention of delamination of ceramic tiling on exterior walls has been an important research theme in the building finishing technology field, and it has been extensively studied. Recently, delamination occurs mainly because of an adhesion failure of the interface between the concrete substrate and the mortar. Given

this situation, some new application methods have been researched and developed. One of these methods is the application of a newly developed organic elastic adhesive.

Japan's Ministry of Construction has carried out joint research on this topic in cooperation with the private sector. The purpose of the research was to collect data for exterior ceramic tile finishing and stone finishing adhesion systems using organic adhesives. The research also determined quality criterion for the organic adhesives, performance evaluation guidelines for exterior tile finishing panels and stone finishing precast concrete panels using the organic adhesives and standard construction specifications for exterior ceramic tile finishing and stone finishing systems with organic adhesives.

In the research project, the following 3 research themes were investigated.

1. Evaluation of adhesives and subsidiary materials for exterior ceramic tile finishing and stone finishing systems
2. Development of exterior ceramic tile finishing and stone finishing manufacturing systems
3. Development of on-site exterior ceramic tile finishing and stone finishing construction systems

The main results of the research project were described in this paper.

2 Types of organic adhesives and their quality

2.1 Types of organic adhesives

2.1.1 Classification for use

Classification for use of the organic adhesives is shown in Table 1.

2.1.2 Main components in the organic adhesives

Table 2 shows a classification of the main components in the organic adhesives by resin type.

Table 1: Classification for use of the organic adhesives

Classification	Use
Exterior use	Type 1 The organic adhesive is independently used
	Type 2 The organic adhesive is used with metallic fasteners

Table 2: Main components of the organic adhesives

Types of resin	Symbol	Type
Epoxy resin	EP	2 components
Urethane resin	PU	1 component
		2 components
Silicone resin	SR	1 component
		2 components
Modified silicone epoxy resin	MS-EP	1 component
		2 components

2.2 Quality of organic adhesives

Quality of organic adhesives must be as follows.

1. It must be uniform in appearance and contain no harmful substances.
2. It must not be harmful to the ceramic tile, stone and substrates which comes in contact.
3. It must not include organic solvents which are regulated by law or other applicable regulations.
4. It must maintain its quality even when left in temperature of 20 ± 15 degrees centigrade and relative humidity of $65 \pm 30\%$ for 6 months after manufacturing.
5. It must clear all test items in Table 3 indicated with X. The tests are applied determined methods. It must also be fitted for criterion as shown in Table 4.

Table 3: Test items for organic adhesives

Items	Type 1	Type 2
The end mixture	X	X
Storing stability	X	X
Adhesion strength		
Standard condition	X	X
Curing at 5 ± 2	X	X
Alkali solution immersion	X	X
Repeated of heating & cooling	X	X
80 ± 2 heating	X	X
Film strength		
Standard condition	X	X
Curing at -20 ± 2 degrees centigrade	X	X
Curing at 80 ± 2 degrees centigrade	X	X
60 ± 2 degrees centigrade		
Alkali solution immersion	X	X
80 ± 2 degrees centigrade heating	X	X
Heat resistance	X	--
Slip resistance	X	--

Table 4: Quality criterion for the organic adhesive

Items	Criterion (Unit: N/mm ²)
Storing stability	No remarkable change of volume or viscosity
The end mixture	Clear color at end mixture
Adhesion strength	
Standard condition	Over 0.60 and a CFR over 70%
Curing at 5±2 degrees centigrade	Over 0.40 and a CFR over 50%
Alkali solution immersion	Over 0.40 and a CFR over 50%
Repeated of heating & cooling	Over 0.40 and a CFR over 50%
80±2 degrees centigrade heating	Over 0.40 and a CFR over 50%
Film strength	
Standard condition	Over 1.0
Curing at -20±2 degrees centigrade	Over 1.0
Curing at 80±2 degrees centigrade	Over 1.0
60±2 degrees centigrade	
Alkali solution immersion	Over 1.0
80±2 degrees centigrade heating	Over 1.0
Film elongation	
Standard condition	Over 30%
Curing at -20±2 degrees centigrade	Over 30%
Curing at 80±2 degrees centigrade	Over 30%
60±2 degrees centigrade	
Alkali solution immersion	Over 20%
80±2 degrees centigrade heating	Over 20%
Heat resistance	Stable against 9.80N weight at 80 degrees centigrade for 4 weeks
Slip resistance	No slipping

CFR: Cohesion failure ratio

Even if adhesion strength is under this numerical value, it is judged suitable in cases of cohesion failure in substrates, ceramic tile or stone.

3 Outdoor exposure test

The outdoor exposure test is described as a representative result of the research in this paper.

3.1 Factors and levels of experimentation

The factors and corresponding attributes of the experiments are shown in Table 5.

Table 5: Factors and aspects of experiments

Factors	Attributes
Exposure sites	Cold district: Monbetsu (Hokkaido) General district: Tsukuba (Honsyu island)
Substrate	Precast concrete panel
Exposure angle	90° (Vertical plane from ground)
Direction	South
Adhesives	Organic (elastic type) 12 kinds MS-EP:9, PU:2, EP:1 MS-EP:9, PU:2, EP:1 Organic (hard type epoxy resin) 1 kind EP Polymer (SBR latex) cement mortar 1 kind PCM
Ceramic tile	45X95mm With/Without back keys 60X227mm With/Without back keys 300X300mm
Application method	Coating adhesive with trowel & pressing on the substrate
Evaluation	Pull-off test

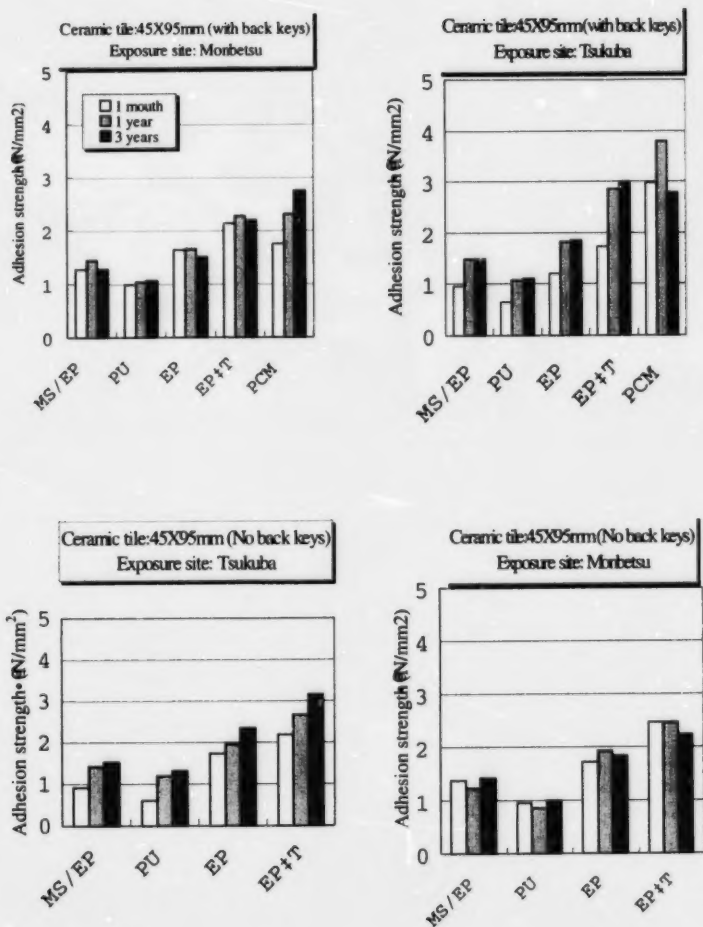
3.2 Test results

Adhesion strength change during the elapsed time is shown in Fig. 1. The adhesion strength of elastic organic adhesives one of the objectives of this research is less than the adhesion strength of hard type epoxy resin adhesive or polymer cement mortar.

However, it was not determined that their strength deteriorates in accordance with elapsed time and there were no differences between exposure sites with a comparison between the adhesion strength of organic elastic adhesives with elapsed time and exposure sites.

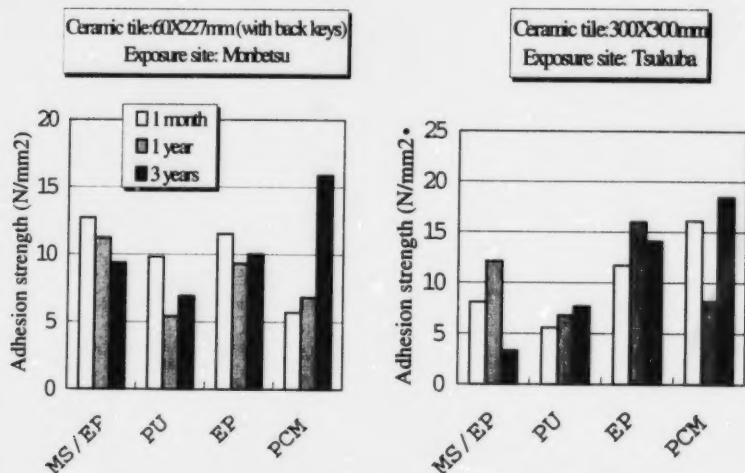
The most common cohesion failure was in adhesives or substrates. However, adhesion failure between organic adhesives and substrates was only partially observed.

Therefore, adhesion durability must be further evaluated under exposure tests for a longer period of time.



Types of adhesives are indicated with symbols which are shown in Table 2.

Fig. 1(1) : Adhesion strength change with elapsed time



Types of adhesives are indicated with symbols which are shown in Table 2.

Fig. 1(2) : Adhesion strength change with elapsed time

4 Practical use of the research results

In addition to the exposure test, the following experimental evaluations were carried out in the research project.

1. Adhesion strength of the organic adhesives
2. Physical properties of the organic adhesive film
3. Usable time and continuous paste condition
4. Slip resistance
5. Accelerated weathering of tile laminated panels
6. Repeated of heating and cooling of tile laminated panels
7. Repeated of freezing and melting of tile laminated panels
8. Sheer strength of tile laminated panes
9. Tension and sheer strength of metallic fasteners
10. Stone contamination by organic adhesives
11. Fire resistance of tile laminated panels
12. Shock-resistance of tile laminated panels
13. Zero span tension resistance of tile laminated panels
14. Relationship between application method and adhesion

Based on these experimental results, quality criterion for the material, performance evaluation guidelines for exterior tile laminated panels and stone laminated precast concrete panels using the organic adhesives are proposed. Standard construction specifications for exterior ceramic tile finishing and stone finishing systems with organic adhesives are also proposed. These guidelines and specifications can found in a document prepared by Japan's Ministry of Construction entitled: 'Guideline for Repair Work Construction Management'.

5 Conclusion

Application of a newly developed organic elastic adhesive as a way to prevent delamination of exterior ceramic tiling was taken up as a research project theme. The representative material, which consists of modified silicone and epoxy resins, has a large capacity for transformation after curing. In this study, the organic adhesives and adhesion systems were experimentally estimated. As a result of this study, quality criterion for the material, performance evaluation guidelines for exterior tile finishing panels and stone finishing precast concrete panels using the organic adhesives and standard construction specifications for exterior ceramic tile finishing and stone finishing systems with organic adhesives have been proposed.

6 Acknowledgements

The research committee for this project worked in cooperation with the Japan Association for Building Research Promotion, and the authors would like to express their appreciation to the numerous other research members who participated.

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80 A NEW TYPE OF CLEAR COATING SYSTEMS FOR WOOD

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Abstract

Four types of clear coating systems for wood were evaluated through sunshine carbon arc irradiation, outdoor exposure, and exposure at elevated temperature. Color difference, residual gloss, and specific reflection spectra within visible rays were measured during these aging tests. Among the tested systems, distinguished durability was found for a new type of coating system, which includes a new type of fluoropolymer curing at room temperature as a top coating material and polyethylene-glycol methacrylate (PEGMA) solution as a primer. Color change mechanisms in three types of aging tests were also discussed in the study.

Keywords: Accelerated aging, clear coating, fluoropolymer, natural aging, polyethylene-glycol methacrylate, wood

1 Introduction

Wood clear coating is popular in Japan because texture of wood is not lost by coating. Although polyurethane clear coating is often used for this purpose, peeling and cracking usually appear in an early stage. According to the standard coating specification established by the Architectural Institute of Japan, none of the wood clear coating systems with the exception of the semi-transparent pigment staining has been recommended for exterior use.

In this context, a new clear coating system that includes a new type of fluoropolymer coating as top coating and polyethylene-glycol methacrylate (PEGMA) as primer was evaluated in order to compare it with an existing polyurethane clear coating system. The fluoropolymers used in the clear coating

systems are alternate copolymers of fluoro-olefins and vinyl ethers (Symes and Conti-Ramsden 1986) and they can be cured at room temperature by mixing with isocyanate compounds. The PEGMA shown in Fig. 1 is soluble in water and the PEGMA solution was used in the substrate preparation step to improve stability of wood and compatibility of wood with coating materials.



Fig. 1: Molecular structure of polyethylene-glycol methacrylate (PEGMA)

2 Experimental

2.1 Specimen

Douglas fir (*Pseudotsuga menziesii*) sized 75mm(R) x 150mm(L) x 20mm(T) was used as substrate. Four types of coating systems listed in Table 1 were applied to the planned substrate.

Table 1: Tested clear coating systems

Coating system	Coating material	Coating rate (Kg/m ²)
A	1) PEGMA aqueous sol.	0.1 - 0.15
	2) Polyurethane sealer	0.06 - 0.08
	3) Polyurethane sanding sealer	0.06 - 0.08
	4) Polyurethane sanding sealer	0.06 - 0.08
	5) Fluoropolymer top coating	0.06 - 0.08
B	1) PEGMA aqueous sol.	0.1 - 0.15
	2) Polyurethane sealer	0.06 - 0.08
	3) Polyurethane sanding sealer	0.06 - 0.08
	4) Polyurethane sanding sealer	0.06 - 0.08
	5) Polyurethane top coating	0.06 - 0.08
C	1) Polyurethane sealer	0.06 - 0.08
	2) Polyurethane sanding sealer	0.06 - 0.08
	3) Polyurethane sanding sealer	0.06 - 0.08
	4) Fluoropolymer top coating	0.06 - 0.08
D	1) Polyurethane sealer	0.06 - 0.08
	2) Polyurethane sanding sealer	0.06 - 0.08
	3) Polyurethane sanding sealer	0.06 - 0.08
	4) Polyurethane top coating	0.06 - 0.08

2.2 Sunshine carbon arc irradiation test

A sunshine carbon arc irradiation test (ISO 4892-4:1994) was used as a basis to subject the coated specimens to accelerated aging. Water spraying was applied for 18 minutes for every 120 minute cycle. Irradiation intensity (wave length range: 300-700nm) was often monitored at the position of the specimen surface by use of an accumulated photometer (Suga Test Instrument Co., Ltd., PH-52-S). The test was carried out for 2080 hours.

2.3 Exposure test at elevated temperature

Coating system A and B listed in Table I were evaluated by the exposure test at elevated temperature. The specimens were exposed in the chamber controlled at 63 C which is same as the black panel temperature in the sunshine carbon arc irradiation test. The test had been conducted for 1000 hours. The specimens were wrapped with aluminium foil in order to avoid the influence of light.

2.4 Outdoor exposure test

The specimens were mounted on the rack at an angle of 30 degrees facing south in the outdoor exposure site of the Building Research Institute (Japan). The specimens had been exposed for one year since June 1986.

2.5 Measurement of color difference, gloss, and specific reflection spectra

Color difference and gloss were measured every 60 - 120 minutes in the carbon arc acceleration test and the exposure test at 63 C. These values were also measured every month in the outdoor exposure test. Further, specific reflection spectra in visible rays (360-720 nm) were measured every 120 hours in the acceleration test and the exposure test at 63 C and measured every month in the outdoor exposure test.

3 Results and discussion

3.1 Changes in appearance

Through visual inspection, micro-cracking could be observed in the polyurethane top coating layer of system B and D after about 1500 hours of carbon arc irradiation. On the other hand, there was no micro-cracking in the fluoropolymer top coating layer of system A and C. Peeling and delamination of coated layers occurred after about 1500 hours' irradiation in coating system C and D, which did not include a PEGMA application process.

These test results suggest that fluoropolymer application to top coating improves durability of coated layers and that application of PEGMA imparts better compatibility between coated layers and substrate. This improved compatibility was separately confirmed in a cross-cut test whose results were not shown in this paper.

3.2 Changes in color difference

Fig. 2 shows the color difference values obtained from the acceleration test. The color difference values in the coating systems increased in three different patterns up to 1000 hours of testing. After 1000 hours, all systems reached almost the same

values however, after 1500 hours the values showed wide variations in color difference. These variations are mostly due to unequal partial peeling, delamination, and other defects. Therefore, only the increasing patterns of color difference values up to 1500 hours are discussed for the systems reported here.

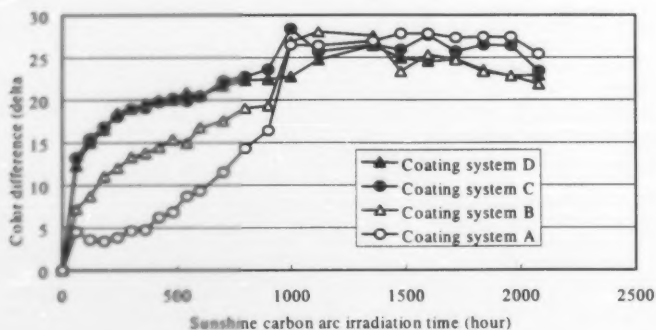


Fig. 2: Color difference changes in the sunshine carbon arc irradiation test

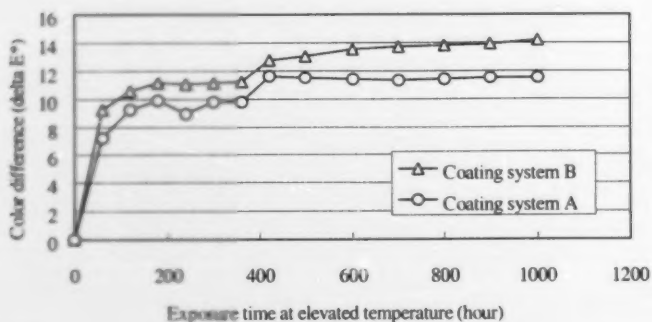


Fig. 3: Color difference changes at the elevated temperature

The increasing patterns up to 1000 hours for system C and D were similar, and the values increased rapidly in early stage. On the other hand, the color difference values of system A and B increased more gradually than system C and D. In particular, system A that showed the slowest increase among all the systems tested. If the values after 1000 hours irradiation were compared, all the systems showed similar values. This suggests that there are certain limitations to the extent one can control such effects in regards to color difference of the coating systems.

In order to investigate the effects of elevated temperature, system A and B were also exposed at 63°C as shown in Fig. 3. It was recognised that color changes could be accelerated by exposure at elevated temperature. It was also recognised that system A and B showed similar behavior. These results are different from those obtained of samples exposed to the sunshine carbon irradiation test.

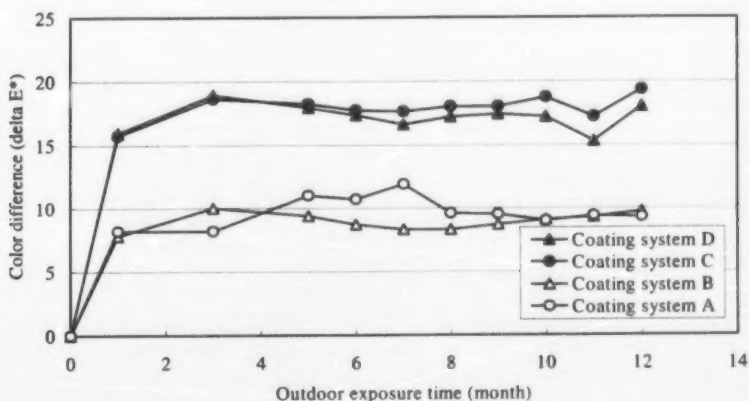


Fig. 4: Color difference changes in the outdoor exposure test

Table 2: Comparison of light irradiation intensity between the outdoor exposure test and the sunshine carbon arc irradiation test

Test	Irradiation intensity (Ultraviolet rays)	Irradiation intensity (Visible rays)
Outdoor Exposure at Building Research Institute	5.1 W/m ² =1.6x10 ⁸ J/m ² /year (wave length:305-395nm)	6.7x10 W/m ² =2.1x10 ⁹ J/m ² /year (wave length:395-715nm)
Sunshine carbon arcs irradiation test	7.6x10 W/m ² =2.7x10 ⁸ J/m ² /1000 hours (wave length:300-400nm)	1.9x10 ² W/m ² =6.8x10 ⁸ J/m ² /1000 hour (wave length:400-700nm)

Fig. 4 shows the color changes of samples exposed to the outdoor exposure test. System A and B, that include PEGMA application step, showed less color changes than system C and D. In the acceleration test, system A and B reached the same color difference values after 1000 hours irradiation. However, they kept smaller color difference values than system C and D after one year of outdoor exposure. If the acceleration test is consistent with the outdoor exposure test, the color difference values of all the systems should reach almost same level also in the outdoor exposure test. Such outdoor exposure duration should correspond to 1000 hours irradiation in

the acceleration test.

To discuss such possibility, the color difference values in both tests were compared based on the amount of irradiation energy. Table 2 shows light irradiation intensity measured in the outdoor exposure test (Nireki et al. 1988) and the sunshine carbon irradiation test. Amounts of ultraviolet and visible light in the outdoor exposure test and the acceleration test were calculated based on Table 2. The color difference values of system C and D in the both tests were compared at the same irradiation intensity as shown in Fig. 5.

Correspondence between the both tests can be verified from values presented in Fig. 5. It can be seen that the acceleration test shows higher color difference values in the range where there is a large amount of irradiation. The color difference values were also plotted based only on ultraviolet light; no correspondence is observed here either.

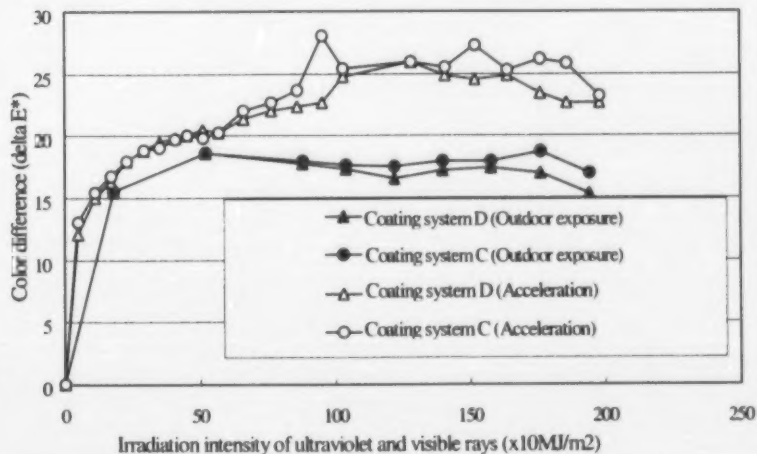


Fig. 5: Color difference changes vs. irradiation intensity.

3.3 Comparison of color change mechanisms by specific reflection spectra in visible rays

As mentioned above, the coating systems were compared through the outdoor exposure test, the exposure test at 63 C, and the acceleration test. System A and B showed fewer color changes in every test because of PEGMA application; however, it is considered that the increasing patterns of the values could not be reasonably consistent through all the tests.

Color changes of specimens can occur due to infrared, visible or ultraviolet radiation, and other factors. As it is difficult to separate these factors in the study, specific reflection spectra were compared to check a color change mechanism in each test. If the mechanisms are similar, the changing patterns in the reflection spectra should be also similar.

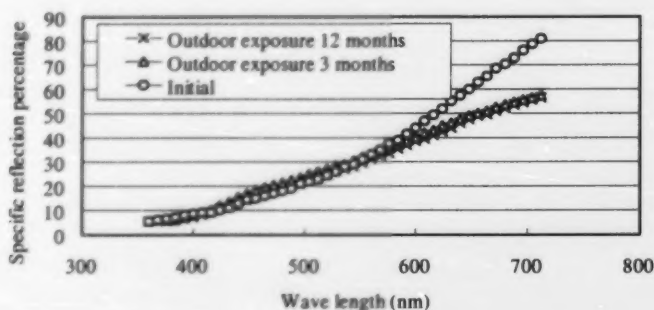


Fig. 6: Changes of specific reflection spectra in the outdoor exposure test for coating system A

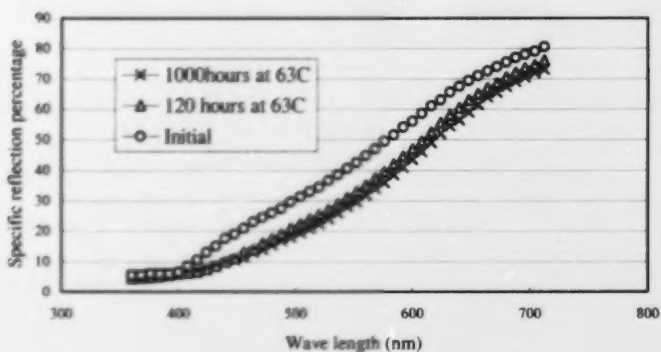


Fig. 7: Changes of specific reflection spectra in the exposure test at 63 C for coating system A.

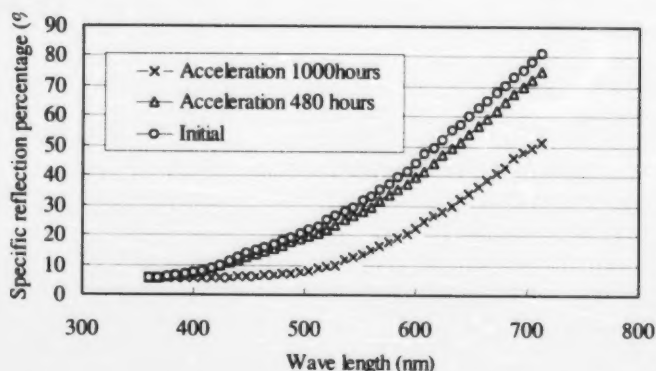


Fig. 8: Changes of specific reflection spectra in the acceleration test for coating system A

Fig. 6 to Fig. 8 show the changes in specific reflection spectra for system A. As shown in the figures, the changing patterns were clearly different among the three tests. This fact suggests that the color change mechanisms are different among the three tests.

3.4 Gloss retention

Fig. 9 and Fig. 10 show the changes in gloss retention percentages in the acceleration test and the outdoor exposure test. Although the figure was not shown in the paper, the gloss retention percentages in the exposure test at 63°C were not changed throughout the test. As shown in Fig. 9, system A and C, in which the fluoropolymers were used for top coating, showed higher gloss retention percentages up to 2000 hours irradiation in the acceleration test. System A and C showed higher gloss retention values also in the outdoor exposure test as shown in Fig. 10.

The gloss retention percentages in both the acceleration test and the outdoor exposure test were also plotted based on amount of irradiated ultraviolet energy. The good accordance was not recognised between the outdoor exposure test and the acceleration test. The gloss retention values in the outdoor exposure test were lower than those in the acceleration test, if they were compared at same amount of irradiation.

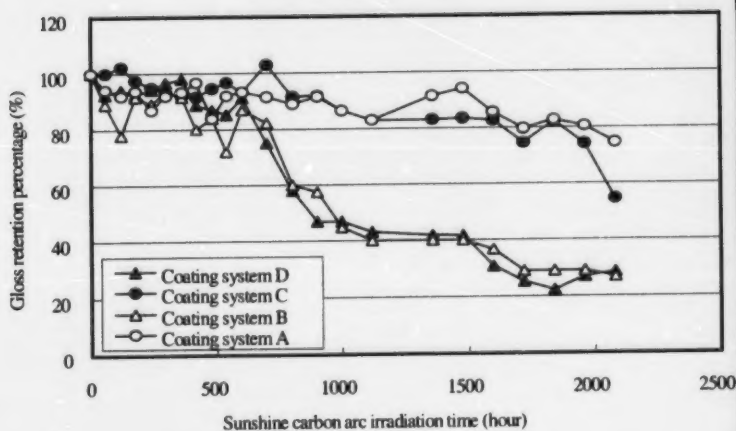


Fig. 9: Changes of gloss retention value in the acceleration test

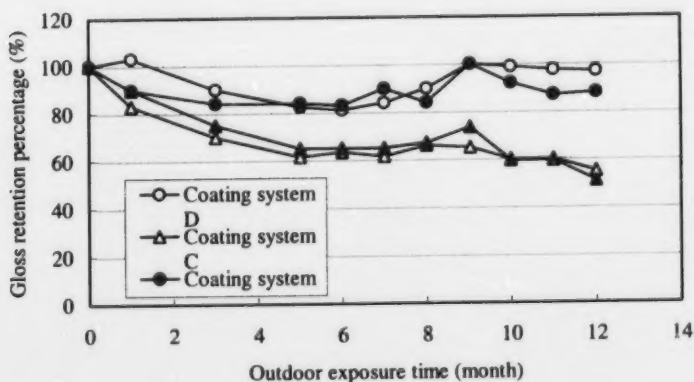


Fig. 10: Changes of gloss retention value in the outdoor exposure test

4 Conclusions

Application of PEGMA solution in substrate treatment process and application of new type of fluoropolymers as top coating materials for wood clear coating systems were evaluated in this study. Durability evaluation was performed through outdoor exposure, sunshine carbon arc irradiation, and exposure at elevated temperature. Color and gloss changes of the specimens were measured periodically. Effects of PEGMA solution and fluoropolymers were investigated. Furthermore, specific reflection spectra in visible light range were measured to compare the color changing mechanisms in the deterioration tests. The results obtained could be summarized as follows;

Application of PEGMA solution imparted the controlling effects of color changes. The effects were confirmed up to 1000 hours in the acceleration test. The effects were also confirmed in the outdoor exposure test and the exposure test at the elevated temperature; however, it is difficult to predict the long term behavior because color changing mechanisms are different among the deterioration tests.

The color changing mechanisms in the deterioration tests were verified by measuring the specific reflection spectra in the visible light range. Changing patterns of the spectra were different in the tests. It seems that a comparison of specific reflection spectra is useful to verify whether or not color changing mechanisms are similar.

It was confirmed that new types of fluoropolymers were useful for wood clear coating systems. Such systems could keep their initial gloss up to 2000 hours in the acceleration test. On the other hand, traditional systems with polyurethane top coating lost their initial gloss after several hundreds hours exposure.

As to the color changes, the color difference values in the outdoor exposure test and the acceleration test were compared based on amount of ultraviolet and visible radiation energy. The accordance between the two tests could not be observed.

As to the gloss retention, similar plots based on amount of irradiation energy were investigated. The agreement between the two was not evident. The gloss retention values in the outdoor exposure test were lower than those in the acceleration test if they were compared at same amount of irradiation.

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81 SERVICE LIFE PREDICTION OF POLYMERIC BUILDING MATERIALS USING THE "ACID DEW AND FOG TEST"

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Abstract

Transparent acrylic sheet materials for sporting halls and several elastomeric building sealants were tested to prove the influence of acid atmospheric precipitations on the aging process in service. Exposure was carried out using the ADF test (Acid dew and fog test) which is a new accelerated artificial weathering test including the simulation of acid atmospheric precipitations. The Olympic sporting halls of Munich have shown that the first of the acrylic roof plates have already reached the end of their service after few years while others have retained good appearance beyond 26 years in service. This huge difference in the material lifetime could not be predicted by use of the current artificial weathering tests. Examining specimens of the aged, strongly damaged roof plates and specimens of the new replacement materials proved that acid depositions contribute significantly to the photochemical aging of poly(methyl methacrylate). Simulated acid dew and fog droplets were found to initiate crazes on the material surfaces leading to the kind of damage, observed in practice. A significant interaction of UV radiation, water content and acid depositions was found which must be considered in designing the test. Examining six typical types of elastomeric building sealants proved that the speed of the photochemical aging as well as the durability rankings are significantly influenced by the presence of acid depositions on the material surface during weathering. Compared with experience made in the field the ADF test results correlate better to the practice than the results from current weathering tests. The ADF test proved to be a valuable tool in predicting the durability and lifetime with satisfying accuracy and reliability.

Keywords: Acid rain, acrylic roofings, photo aging, building sealants, weatherability.

1 Introduction

Acid rain, atmospheric acidity and acid depositions, are phenomena characterising one of the most important environmental problems we have had since the beginning of industrialisation about 100 years ago, the acidification of our environment. The question asked here, do we still have acid rain, must be clearly answered with a yes (Möller 1998). Acid rain and other acid precipitations like dew, fog or dust are formed when specific pollutants combine with water or dust particles in the atmosphere. By far the largest contributor is sulphur dioxide (SO_2), a by-product of fossil fuel combustion which, when oxidised to H_2SO_4 , accounts for 62–77% of the acidity in rain. Most of the remaining acid is from nitrogen oxides (NO_x), found mainly in automobile exhaust, which are oxidised to HNO_3 (15–32%). In addition to these two main species, HCl (2–16%), also produced from burning of coal, is found in the atmosphere (Rodgers et al. 1998). A complicating factor in the study of the effects of acid precipitations on materials is that the severity of the exposure varies greatly with time and location. The effect of acid precipitations as promoter of the metal corrosion has been known and has been investigated widely. The influence on the lifetime of polymeric materials in outdoor use is less investigated. Car manufacturers have made the experience that automotive finishes undergo a rather fast degradation in weathering sites which are characterised by frequent occurrence of acid rain and other acid depositions combined with high UV irradiation, high temperature and high humidity like the area around Jacksonville, FL, presently used as place for storage of imported cars waiting for distribution into the south of the USA. A synergistic effect of UV radiation and acid depositions was found as reason for the accelerated degradation (Schulz and Trubiroha 1996). Therefore, a great deal of work is being performed on detecting appearance damage and on developing acid resistant coatings. But this extensive work in automotive coatings is unlikely to meet the needs of the building industry with regard to long life and survivability, because polymeric constructive materials are different from car finishes. The influence of acid precipitations on the service life of polymeric building materials may be underestimated. In result of investigations in constructive polymeric materials for automotive accessories Schulz and Trubiroha 1997 concluded that it can be expected that aggressive air pollutants or their product are also responsible for the unexpected damage or phenomena on polymeric building materials observed in practice. So, predictions of the material durability and lifetime made by use of the current weathering tests for polymeric materials only seldom correspond to the experience made in the field. For instance, the different service life of polymeric building elements observed in differently industrialised areas made conclude that aggressive air pollutants contribute to the polymer degradation during outdoor use.

2 The aim of the examinations

The aim of this paper was to examine whether acid atmospheric precipitations influence the photodegradation of polymeric building materials. In order to predict the durability and lifetime of the materials with satisfying accuracy and reliability a further objective of the work was creating the basis for developing a new accelerated artificial weathering test method tailored to meet the crucial climatic conditions for the special building material and the special application.

Examinations carried out with two typical examples are presented in this work.

3 The simulation of acid precipitations

During investigations of the appearance durability of modern automotive coatings in air polluted weathering sites it was found that weather episodes where acid depositions are dried in the sunshine are most responsible for the so-called environmental etching observed in the field. Simulating the mutual action of UV irradiation and acid in laboratory a new accelerated artificial test was developed and applied for testing of automotive and aircraft coatings successfully. This new weathering test introduced as "Acid Dew and Fog test" (in short ADF test) differs from the current artificial weathering tests in that acid precipitations are included in the weathering cycle. The ADF weathering cycle is characterised by continuous UV radiation and varying climatic conditions as shown in table 1. Usually, after five of such cycles two other cycles are applied in the same way but without acid spray. So, the coating surfaces are affected simultaneously by acid and UV radiation for about 40 % of total exposure period. Acid dew and fog are simulated by using a mixture of sulphuric, nitric and hydrochloric acid (mass ratio 1 : 0,30 : 0,17), the most essential acid ingredients found in atmospheric precipitations, diluted to different pH-values (1,5, 2,5). The ADF test were performed using a fluorescent UV lamp weathering device "Global-UV-Test". Spectral irradiance was according to ISO 4892-3:1994, "fluorescent UV lamp combination" with an UV irradiance of 45 W/m².

Table 1: The 24h ADF cycle.

1	A short spraying of the simulated acid dew or fog onto the surface of the vertically orientated test specimens at 35 °C (about 3 mg of solution per 1 cm ² exposed coating surface)
2	14 h dry period at different climatic conditions (9 h at 35 °C/ 30 % RH, and 5 h at 60 °C/5 % RH)
3	4 h rain period with continuous water spray (demineralised water at 35 °C)
4	6 h dry period at 60 °C/ 5% RH.

4 Transparent acrylic sheet material (PMMA)

Poly(methyl methacrylate) (PMMA), because of its good environmental resistance is used extensively outdoors where transparency and light weight are required. The material does not yellow like polycarbonate and is much more scratch resistant. Impact resistance of PMMA is much less than that of polycarbonate but is greater than that of glass. Therefore, building applications as transparent roofings are wide-spread. Like all other polymeric materials PMMA undergo photochemical aging. A commonly observed failure mode is the gradual development of crazes. Loss of transparency or becoming opaque characterise usually the end of the material's lifetime. Comparing the aging state of the acrylic roofings of the Olympic sporting stadiums in Munich and Berlin there is a great difference in the number of the roof plates which have lost their transparency although the roofs of both stadiums have about the same age of 26 years and were made from the same material. Figure 1 shows a roof plate with the typical flaky damage, which must be replaced because of bad appearance. Differences in industrialisation between the two sites might be a reason for the influence of the location. Although the average values of the acidity of the atmospheric precipitations do not differ significantly between the two urban areas data about depositions collected near Munich (Bayerisches Landesamt für Umweltschutz 1998) revealed that acid precipitations occur sporadically up to three times a year with an acidity exceeding the yearly average values thousandfold and more. To examine the contribution of these stronger acid precipitations on the aging process in combination with the other climatic conditions investigations were carried out with specimens of the aged, strongly damaged roof plates from the Munich Olympic stadium and specimens of the new material for replacing the plates which had to be removed. The material is known as Plexiglas 215 (stretched in two axis) meeting

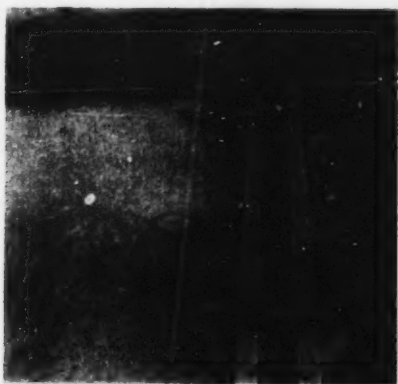


Fig. 1: A 25 years old damaged PMMA roof plate

the flammability requirements of DIN 4102-B2 standard. A recent publication (Lehmann 1997) describes the frequent changes of wetting and drying as most responsible for arising the damage of the weathered roofing side. The almost clear surface parts which can be seen in Figure 1 near the metallic disks or frames being necessary for fixing the plates could be evidence for this claim because these surface parts were mostly warmer then the other parts in clear nights and therefore, fewer and shorter affected by dewing. To prove this influence an almost clear specimen with only few crazes was cut from the plate shown in figure 1 and exposed to wet/dry cycles consisting of 96 h inserting in water (23°C) and 96 h storage in vacuum (23°C, 1 kPa). All crazes which could be realised on the specimen before the test changed into the typical flaky damage. But this procedure could not provoke new crazes. Arising crazes must have other reasons. Proving acid depositions as a possible reason, a equal specimen was exposed to an ADF cycle as described above and a subsequent wet/dry-cycle (96 h water / 96 h vacuum). Figure 2 shows the specimen after each step of the test procedure. As can be seen each droplet of the simulated acid dew and fog which can be seen in part B initiate crazes which are start point for the kind of

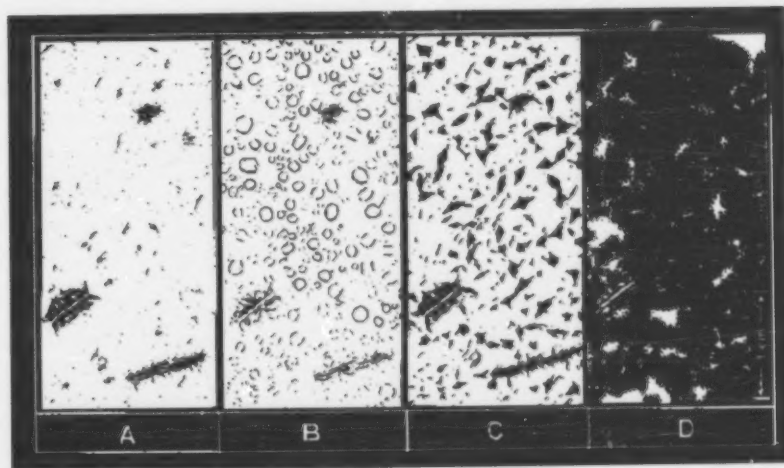


Fig. 2: The action of acid droplets on the naturally aged PMMA roof plate
(A: original, B: after spray with acid solution pH 1.5, C: after ADF cycle, D:
after wet/dry cycle performed additionally)

damage observed in practice. To examine the interaction of UV radiation, time of wetting and acid depositions a completely craze-free specimen including a surface part which was covered by one of the small round metal disks (see figure 1) was taken from the same aged plate and exposed to one ADF cycle with pH 1.5. As can be seen in figure 3 the acid droplets generated crazes only on the part outside the big circle, it means on the surface area which was not protected from the UV radiation during the

service life. It reveals that the acrylic sheet material lose its good initial acid resistance with increasing UV radiation exposure. Up to ten wet/dry cycles with the same specimen performed subsequently could not transform the crazes in the slate-like damage in contrary to the specimen shown in figure 2. Evidently, the less average water content of this specimen during its service life resulted in a slower photochemical aging of the acrylic material. Differences in the mol mass could be found as evidence of different aging states using the GPS-method. In further examinations virgin sheet material was exposed to the ADF test with and without an preceding acid-free artificial weathering in the same weathering device. In result only the at least 42 d pre-weathered specimens reacted with arising of crazes after the ADF test. Therefore, a test designed for predicting the life time of acrylic sheet material has to consider urgently the interaction of all essential environmental conditions.

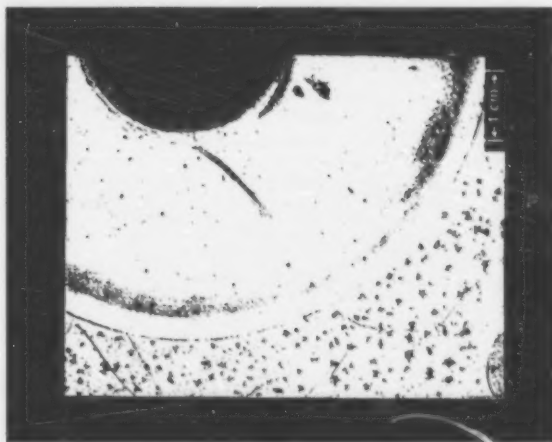


Fig.3: A clear specimen from an natural aged roof plate after one ADF cycle

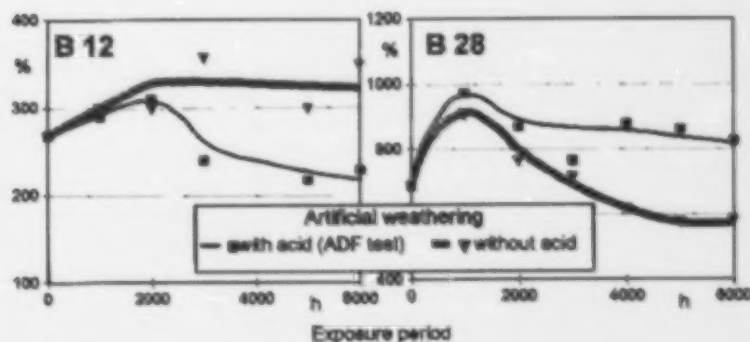
5 Elastomeric building sealants

Building sealants are desired to have the same service life as the building whose joints they have to seal. There is not any experience how acid precipitations influence the sealants' service live and not any reliable testing method which consider this important environmental condition, either. Therefore, examinations were carried out using six different rubbers as foils. They are characterised in table 2.

Table 2: List of the tested elastomeric sealants

No.	Raw material basis	Reaction system	Colour
A 6	Silicone rubber	1 P / basic / Amine-Oxime	grey
A 21	Silicone rubber	1 P / acid / Acetate	white
B 9	Polysulfide rubber	2 P / Manganese dioxide	grey
B 12	Polysulfide rubber	2 P / Manganese dioxide	black
B 28	Polysulfide rubber	Prefabricated	grey
D 24	Polychloroprene rubber	Hot vulcanisation	black

The results of the ADF test performed with an acidic wetting of pH 2.5 were compared with the results from an acid-free artificial weathering test and an outdoor weathering test. The acid-free weathering was carried out in the same way as in the ADF test but without the acid wetting of the specimens. The outdoor weathering was carried out in Leipzig city in an industrial urban environment with a high amount of coal fired domestic heating (Boettger and Bolte 1998). To determine the influence on the essential material properties the tensile strength σ_R and the elongation at break ϵ_R were measured according to ISO 37. Figure 4 compares the change of the elongation at break $\Delta\epsilon_R$ for two different polysulphide rubbers in dependence on the exposure period of the artificial weathering tests with and without simulation of acid precipitations. The figure shows that the presence of acid during weathering can accelerate (see B 12) as well as slow down (see B 28) the photochemical aging.

Fig. 4 Elongation at break $\Delta\epsilon_R$ as a function of the exposure period

In figure 5 the results of the two artificial weathering tests after an exposure period of 3000 hours are compared with the five year outdoor exposure results in terms of the change of elongation at break. The ADF test shows a very good ranking correlation whereas the acid free weathering test do not correlate with the practice. It proves acid

precipitations influence the durability ranking significantly. As can be seen in table 3 an exposure period of 3000 hours seems to be long enough to distinguish the tested samples in terms of their behaviour in air polluted environment sufficiently. Concerning the time-compressing effect of the laboratory test the rubbers A 21, B 9, B 12 and D 24 show about the same change of the elongation at break after 3000 h ADF test and 5 years outdoor weathering. No acceleration factor can be defined for the rubbers A 6 and B 28 evidently because of an postcuring effect. No influence could be found on the change of the tensile strength at break $\Delta\sigma_R$. In this case both laboratory tests show the about same good practice correlation (see table 3).

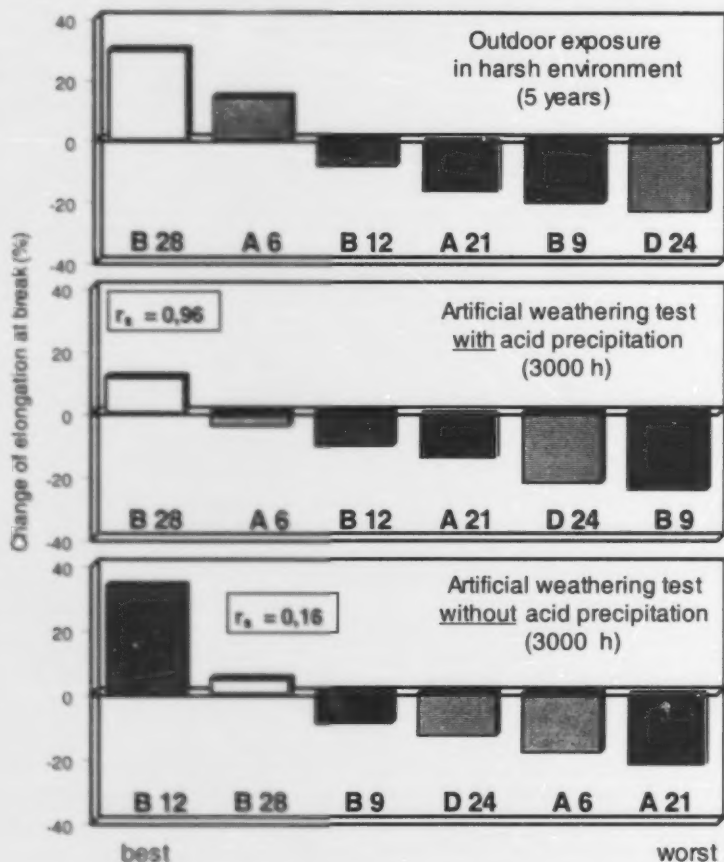


Fig. 5: Comparison of laboratory and outdoor test results for different sealants
(r_s = Pearson ranking correlation coefficient to the outdoor weathering)

Table 3: Pearson coefficients to the 5 year outdoor weathering

Exposure period	ADF test		acid-free weathering	
	for $\Delta\epsilon_R$	for $\Delta\sigma_R$	for $\Delta\epsilon_R$	for $\Delta\sigma_R$
2000	0,80	0,93	0,56	0,88
3000	0,96	0,89	0,16	0,84
6000	0,96	0,89	0,20	0,84

6 Conclusions

Acid atmospheric precipitations influence the photochemical aging of polymeric building materials significantly. They can accelerate as well as slow down the polymer degradation in interaction with the solar UV irradiation and other climatic conditions. Generally, acid precipitations influences the durability ranking which is very important for selecting the best qualified material for the special application. The ADF test whose results showed a better practice correlation as those of current artificial weathering tests can be seen as a first step in designing a laboratory test method which meet best the special requirements of the building material and the special conditions of its use.

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82 THE EFFECT OF HEAT AGING ON ACRYLONITRILE-BUTADIENE-STYRENE (ABS) BLENDS

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Abstract

Degradation of thick ABS materials at elevated temperatures is observed to be predominantly a surface effect which deteriorates the bulk mechanical properties under impact conditions. It is postulated that a critical level of surface degradation initiates polymer crazing in the rubbery butadiene phase, which propagates through the polymer, causing failure. A physico-mechanical study of the effects of heat aging on the rubbery phase in ABS reveals an increase in the loss modulus and $\tan \delta$, demonstrating a decrease in the free volume which may be due to cross-linking of free radicals formed by hydrogen abstraction and chain scission. Microscope FTIR spectroscopy of aged ABS surfaces shows that the degree of butadiene unsaturation decreases with time, and that carbonyl and hydroxyl absorbances increase as expected with thermo-oxidative degradation. Degradation of the rubbery polybutadiene phase in ABS may be initiated by hydrogen abstraction from the carbon α to unsaturated bonds, producing hydroperoxide radicals leading to carbonyl and hydroxyl products. Cross-linking is facilitated by the free radicals produced. Heat aging of ABS at elevated temperatures below the glass transition (T_g) of the SAN phase causes physical aging. This aging of the SAN phase provides thermal energy which allows for a structural reconfiguration of the glassy morphology towards an amorphous polymer. In this case, heat aging relaxes the intrinsic molecular stresses and causes a decrease in the initial rate of transition from the glass to the liquid when temperatures approach T_g . It is also possible that the physical aging of the SAN phase in ABS may contribute to the deterioration of the mechanical properties of the polymer.

Keywords: ABS, degradation, durability, heat aging, physical properties, pipe

1 Introduction

Durability prediction of acrylonitrile-butadiene-styrene (ABS) polymers is important in a wide range of polymer applications, including water pipeline systems. A number of factors influence ABS pipe durability, including material composition, processing conditions, operating conditions, environmental weathering, heat aging, installation and notch damage. The availability of a lifetime prediction model for ABS in pipe applications, would allow for accurate selection of polymer material, according to the environmental and operating

selection of polymer material, according to the environmental and operating conditions the pipe will be required to perform in, thus decreasing the number of future pipe failures and maintenance costs.

In polymers such as polypropylene, low density polyethylene and ABS, specific aspects of the microstructure facilitate thermal oxidation. Tertiary substitution of carbon atoms allows hydrogen abstraction by oxygen to be more thermodynamically favourable. Thermal energy provides the activation energy required for the initiation of oxidation, thus accelerating the overall process of degradation. After periods of exposure to heat and oxygen, the mechanical properties of ABS such as impact strength and elongation to break, deteriorate, leading to polymer failure (Wolkowicz and Gaggar 1981). The literature cites various explanations for the degradation of ABS due to heat aging. Several papers state that the thermo-oxidative degradation of ABS is confined to the rubbery polybutadiene phase, while others claim that degradation of properties is due to a combination of physical aging in the SAN phase and oxidation of the polybutadiene phase (Gesner 1965; Shimada and Kabuki 1968; Wyzgoski 1976; Salman 1991). Shimada and Kabuki (1968) proposed a mechanism for the thermo-oxidative degradation of unstabilised ABS film, proposing that degradation occurred in the rubber phase and that the degradation mechanism leads to the formation of hydroperoxides. Degradation was said to occur by hydrogen abstraction from the carbon α next to trans-1,4 and 1,2 unsaturations in the polybutadiene phase producing hydroperoxide radicals. The rate of reaction follows Arrhenius-type kinetics, and the rate constants and reaction orders were determined from the rate of formation of carbonyl and hydroxyl products as monitored by Fourier transform infra-red (FTIR) spectroscopy.

In this study, thick stabilised ABS specimens were aged at elevated temperatures in order to study the effects of heat aging on ABS pipe. The presence of anti-oxidants and UV stabilisers in the ABS was expected to complicate the mechanism of oxidation due to the effects of stabiliser solubility and diffusion. The study does not consider environmental effects such as the migration and leaching of stabiliser in typical outdoor water pipes. Results and observations are based on accelerated heat aging experiments and it is likely that thermo-oxidation of ABS in real applications would be different. Also considered is the localised oxidation at the surface of the specimen due to the limited oxygen diffusion (DLO) in the polymer as discussed by Wise et al. (1997).

2 Experimental

Commercial ABS pipe resin and styrene-acrylonitrile copolymer (SAN), in the form of impact (notched and unnotched) and tensile specimens conforming to ASTM D256 (ASTM 1993) and AS 1145 (Standards Australia 1989), were heat aged at 90°C and 120°C in a Qaltest Solidsat Universal 2000 series cabinet oven. Oven-turbulence was minimal with wind speeds in the range of 1–2 m/s being measured by a hot wire anemometer. Oven heat distribution was measured by a platinum thermoresistor and found to be within a range of $\pm 2^\circ\text{C}$. Specimens were placed on talc-coated glass plates, occupying $\leq 10\%$ of the total oven space, and air circulation was maintained at 4 exchanges/hour, as required by ISO 188 (ISO 1982). Heat aging experiments were conducted for up to 672 hours (at 90°C and 120°C) for notched and at 120°C for unnotched ABS specimens, with samples removed for mechanical assessment every 168 hours. Heat aging of a different unnotched ABS resin was conducted at 100°C using the same procedure.

Instrumented impact analyses were performed using a Radmana impact tester, in accordance to ASTM D256. Modulus and elongation to break percentages were assessed on an Instron 5565 screw-driven tensile tester (5 kN) using pneumatic grips (70 psi grip pressure) according to AS 1145.

To enable analysis of physical properties such as T_g and loss modulus, specimens were sectioned prior to heat aging to the required dimensions for physico-mechanical property assessment. T_g and loss moduli were measured using a Rheometrics Scientific Solids Analyser RSA II DMTA, for both the rubber polybutadiene phase and the rigid SAN phase of ABS after heat aging at 120°C and 90°C.

To monitor the effects of aging on the characteristic infra-red absorptions of vinyl groups in ABS, and to detect the presence of carbonyl and hydroxyl species due to oxidation, a qualitative analysis of ABS aged at 120°C was conducted on a Perkin-Elmer FTIR 2000 with a microscope accessory. Complete microtome cross-sections (20 μm thick) of ABS aged for 672 hours were investigated using transmission mode, with microscope stage control and imaging, allowing for visual identification of the analysed polymer section.

Positron annihilation lifetime spectroscopy analyses were conducted on ABS aged at 90°C and 120°C to investigate the effects of heat aging on free volume.

3 Results and discussion

3.1 Mechanical property analyses

The impact strength of notched (notching occurred after heat aging) ABS as a function of the aging time at 90°C and 120°C is shown in Fig. 1. At 90°C, the loss of impact strength is not significant. At 120°C, a slight constant loss of impact strength is observed. Onset of a brown colouration occurs on the surface of ABS specimens aged at 120°C, which develops as a function of time (Fig. 2). However, for unnotched ABS aged at 120°C, the impact properties decrease rapidly from an initial value of 245 kJ/m^2 to a value of 30 kJ/m^2 , after which the rate of loss is minimal, as shown in Fig. 3. Aging of a second unnotched ABS resin at 100°C reveals a considerably smaller decrease in impact strength, initially from 148 kJ/m^2 to 122.3 kJ/m^2 after 600 hours. It is expected that impact properties of aged unnotched ABS at 90°C would not be significantly affected, although this will be proven in future work. Aging of SAN copolymer at 90°C did not cause any significant decrease in the unnotched impact strength.

The difference in impact strength for notched and unnotched ABS aged at 120°C and the discolouration observed on the ABS surface after aging, indicates that heat aging causes thermo-oxidative degradation at the surface of ABS, which influences the mode of polymer failure. It is suggested that diffusion of oxygen in the aging times considered is limited to the surface due to the polymer morphology, which leads to an increase in polymer density due to cross-linking caused by aging. Cross-linking at the polymer surface leads to surface stress hardening and an increase in modulus. Under impact conditions, micro-cracks initiate in the hardened layer, which propagate through the bulk of the polymer, causing failure. Wolkowicz and Gaggar (1981) state the loss of impact properties is controlled by a critical depth of degraded polymer and once this depth is reached, impact properties degrade rapidly, as is also seen in this work (Fig. 3). It can be seen that the impact properties show more than an 80% decrease at 168 hours for specimens aged at 120°C. At this time the coloured surface layer was approximately 0.06 mm deep. Further exposure did not reduce the impact

strength, even though the depth of the degraded layer continued to increase, it being 0.5 mm after 672 hours. These results are in agreement with Wolkowicz and Gagger (1981), who found the critical depth to be between 0.07 and 0.2 mm.

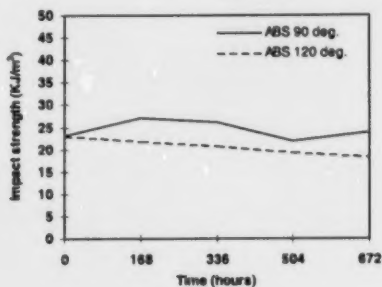


Fig. 1: Impact properties of notched ABS as a function of aging time at 90°C and 120°C



Fig. 2: Colouration of ABS as a function of aging time (left, 0 hours; far right, 168 hours) at 120°C

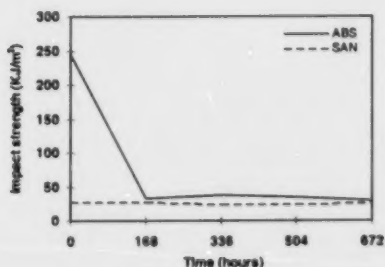


Fig. 3: Impact properties of unnotched ABS as a function of aging time at 120°C, and unnotched SAN copolymer as a function of aging time at 90°C

Examination of the cross-sections of impact specimens after aging at 120°C reveals that the coloration of the ABS was limited to the surface of the specimen, even after heat aging for several months. The literature has cited that coloration in the thermal aging of ABS is due to radical scavengers coupling with peroxy radicals formed during degradation (Faucitano et al. 1996). This would suggest

that the degradation and subsequent discolouration in ABS is confined to the surface of the specimen and is controlled by the rate of oxygen diffusion through the polymer. However, more work needs to be carried out to confirm this, as thermal degradation in the absence of oxygen may be occurring in the bulk polymer. Even so, Fig. 1 shows that notched specimens show a constant failure with time, whereas unnotched specimens show a rapid failure in the first 168 hours due to the effects of the oxidised surface layer. The results indicate that failure is critically dependent on the surface layer of the ABS where crack initiation occurs.

Figure 4 shows the tensile properties of ABS aged at 90°C. It can be seen that the energy to break and elongation to break decrease considerably in the first 168 hours. A slight increase in modulus is also observed. Aging of the SAN copolymer (SAN ratio as in ABS) reveals an increase in the Young's modulus and a slight initial increase in the energy to break and elongation to break, after which these properties are fairly constant (Fig. 4).

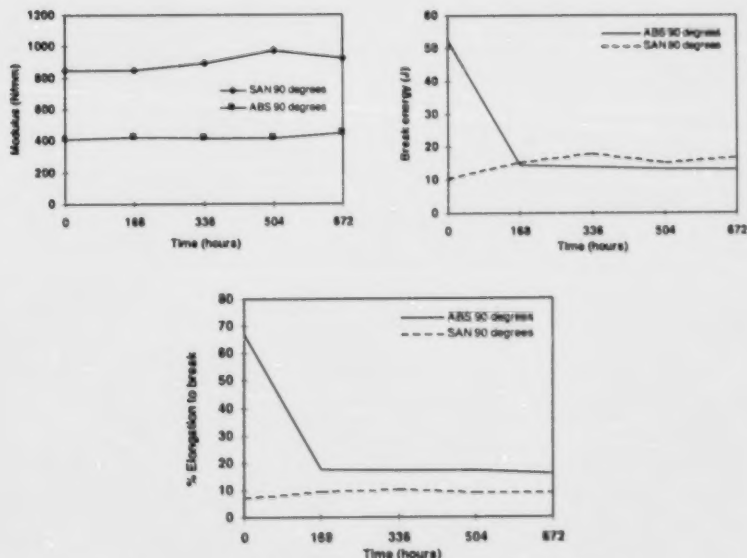


Fig. 4: Tensile properties of ABS and SAN copolymer aged at 90°C

In contrast, as shown in Figure 5, ABS aged at 120°C shows significant decrease in energy to break and elongation to break and an increase in the modulus. Similar decrease in tensile results were reported by Tavakoli (1994) who found a 30–70% reduction in tensile strength for ABS aged at 120°C.

Figure 6 shows a typical stress-strain curve for ABS after aging at 90°C. Although elongational properties decrease, the curve shows a ductile failure mode, indicating that thermal degradation is not severe. As the temperature of aging increases, the ABS shows an increase in modulus and a change in behaviour from ductile to brittle, as shown by the early failure at low elongation to break figures for the ABS aged at 120°C, compared to virgin polymer.

As shown in the first graph in Fig. 4, the increase in modulus for SAN after aging at 90°C is expected. This increase is due to physical aging, as the aging temperature was below T_g of the copolymer. Physical aging of a glassy polymer provides the thermal energy for structural reconfiguration towards an amorphous morphology with reduced free volume due to segmental motion. Aging thus causes a decrease in the initial rate of transition from the glass to liquid when the glass approaches T_g from sub- T_g temperatures. The decrease in free volume causes the actual glass transition temperature to increase, effectively increasing the mechanical modulus. The literature cites that physical aging of polymers causes an increase in polymer density and modulus, and a decrease in impact strength, fracture energy, elongation, creep and the rate of stress relaxation (Wyzgoski 1976; Qi et al. 1993). Initial increases observed for elongation and energy to break of the aged SAN copolymer are attributed to relaxation of intrinsic molecular stresses. Wyzgoski states that the high tensile elongation properties of ABS are also dependent on the deformation of the glassy matrix, so heat aging of the SAN phase would contribute towards embrittlement of the SAN phase and contribute to failure of the ABS polymer (Wyzgoski 1976). Aging of ABS at 90°C shows a decrease in the elongational property, as shown in the second graph of Fig. 5 and Fig. 6, yet as discussed above the SAN phase shows increases in this property when exposed at this temperature, consequently the reductions in elongation shown for ABS must be due to physical changes in the polybutadiene phase.

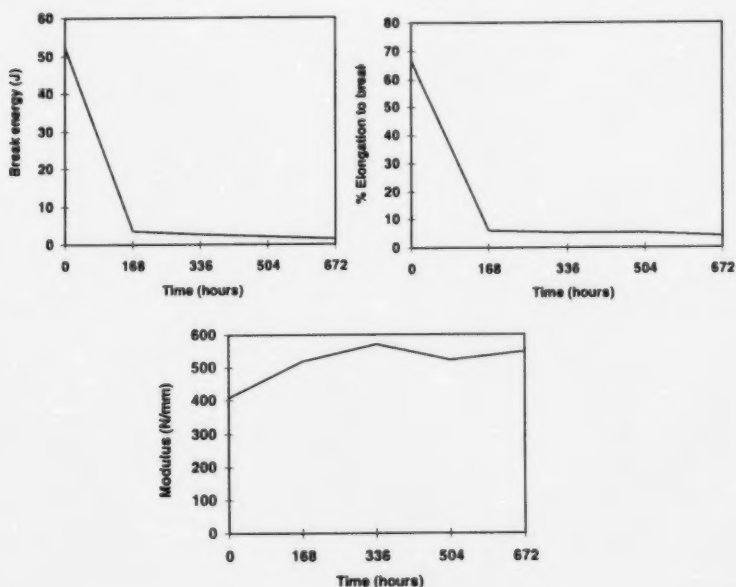


Fig. 5: Tensile properties of ABS aged at 120°C: break energy, elongation to break and modulus

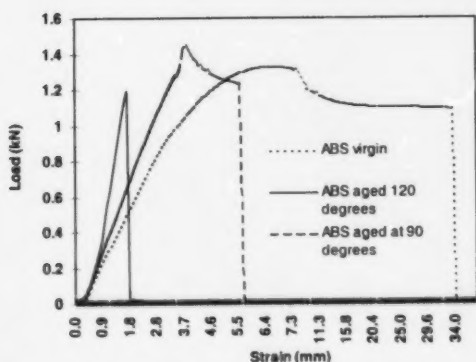


Fig. 6: Typical stress-strain curves for ABS aged at 120°C and 90°C

Brittle failure is observed for ABS aged at 120°C as compared to virgin polymer which shows a ductile failure mode upon tensile testing (Fig. 6). The deterioration of impact and tensile elongation, and the increase in modulus is attributed to thermal degradation of the polybutadiene phase. Colouration on the surface of the polymer indicates that a thermo-oxidative degradation mechanism has occurred causing chain scission and cross-linking of the rubber phase, leading to surface hardening of the aged ABS, which initiates crack formation. It is proposed that thermal degradation in the bulk region of the polymer in the absence of oxygen (surface discolouration is due to oxygen permeability and subsequent reaction with antioxidants) causes chain scission and cross-linking in the polybutadiene phase which deteriorates the bulk tensile properties of the polymer.

The increase in modulus and the decrease in elongation to break property of ABS aged at 120°C and 90°C demonstrate the effects of both surface oxidative and bulk degradation in the polybutadiene phase, which deteriorates the overall mechanical properties. At 90°C the physical aging of the SAN phase does not contribute to the loss of properties of the ABS. At 120°C the effect still has to be analysed for, as shown in Fig. 6, ABS at this temperature distinctly shows brittle failure, compared to a ductile failure for virgin ABS. Consequently the contribution to this behaviour from the SAN and butadiene phases has to be assessed. Future experiments will determine relative contributions as well as whether the thermo-oxidative degradation of ABS has an Arrhenius temperature dependence.

3.2 Physical property analyses

FTIR spectroscopy combined with transmission mode optical microscopy and image analysis, was carried out on a microtomed cross-section of the specimen. This allowed spectroscopic analysis within a localised region of the ABS microtome (100 × 100 μm area). Examination of the specimen surface compared to the bulk of the polymer (Fig. 7), indicates that for ABS exposed at 120°C for 672 hours, thermo-oxidative degradation of the ABS is occurring on the surface but not in the bulk polymer.

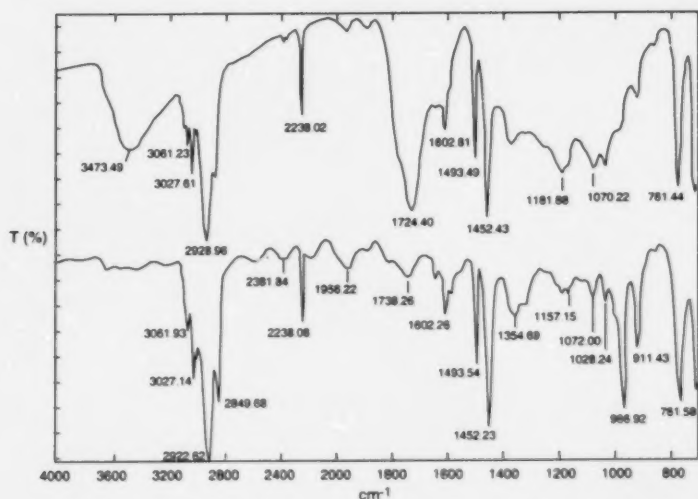


Fig. 7: Transmission FTIR spectra of ABS aged at 120°C. Upper spectrum at the surface and the lower spectrum the bulk ABS polymer

As shown in Fig. 7, this can be explained by thermo-oxidative degradation in the polybutadiene phase of ABS, which causes increases in carbonyl and hydroxyl absorbances and decreases in the absorptions caused by vinyl unsaturations (Gesner 1965; Shimada and Kabuki 1968; Salman 1991). The top spectrum taken from the edge of the sample shows a broad carbonyl band at 1724.4 cm^{-1} and a broad band for the hydroxyl group at 3473.49 cm^{-1} . As discussed, the absorbance bands in virgin ABS corresponding to the trans C=C unsaturation (vinyl) at 966.92 cm^{-1} and the 1,2 butadiene terminal vinyl C-H band being 911.43 cm^{-1} , decrease considerably for the aged ABS specimen.

Dynamic mechanical thermal analyses of ABS specimens heat aged at 90°C and 120°C, and SAN copolymer heat aged at 90°C, were conducted to investigate the effects of aging on the glass transition of the polymer both in the rubber phase and in the rigid SAN phase.

Polymers below their T_g at room temperature are labelled as being glasses and may be annealed or physically aged at temperatures below their T_g . The process of physical aging causes relaxation of intrinsic stresses. This can be seen in a reduced rate of transition from the glassy state to the liquid state, which occurs concurrently with an increase in the glass transition temperature and also an increase in the modulus. Such effects on the SAN phase have been stated to cause embrittlement of ABS polymers. Wyzgoski (1976) showed that physical aging of the SAN phase and embrittlement of the ABS is reversible by heating above the T_g of ABS, indicating that cross-linking is not occurring. The $\tan \delta$ and loss modulus (E'') parameters obtained by DMTA can be used to describe the glass transition of polymers. Figure 8 shows a reduced initial rate of glass transition represented by the $\tan \delta$ parameter, for ABS as the aging time at 90°C increases. Table I shows the E'' and $\tan \delta$ values of the SAN phase of virgin ABS and ABS aged at 90°C. Both parameters increase with aging time, which correlates with the increase in Young's modulus measured by tensile assessment.

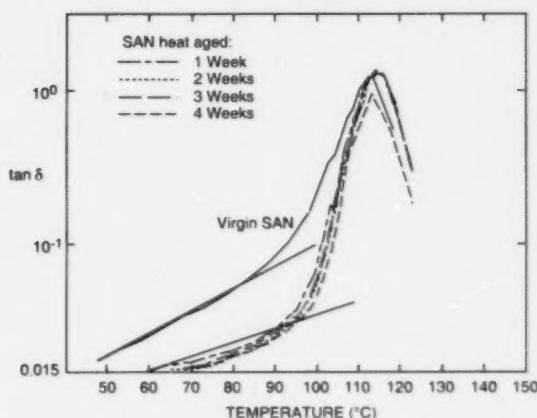


Fig. 8: $\tan \delta$ parameters of the SAN phase of ABS aged at 90°C. Tangents indicate a decrease in the initial rate of transition for aged SAN copolymer

Table 1: E'' and $\tan \delta$ values of the SAN phase of ABS aged at 90°C

Aging period (hours)	$\tan \delta$	Loss modulus (E'')
0	113.02	103.13
168	114.18	108.36
336	114.26	106.77
504	114.26	106.31
672	113.2	107.31

It can be seen in Table 2 that the T_g of the polybutadiene phase at approximately -85°C, does not change as a function of aging time at 90°C. This, together with the data from the mechanical assessment, suggests that significant changes in the morphology and free volume of the polybutadiene phase did not occur, and that the degree of chain scission and cross-linking at 90°C for the aging times considered (672 hours) are minimal. Thus, an explanation for the reduced elongation properties for the ABS aged at 90°C cannot be obtained from this data and further work will have to be carried out to explain this phenomenon.

However, for ABS aged at 120°C, significant differences occur in the glass transition of the polybutadiene phase, reflected by changes in E'' and $\tan \delta$ values as shown in Table 2. Both parameters show an increase with exposure time, indicating a decrease in free volume, possibly due to cross-linking reactions occurring during thermal degradation. This is in agreement with an increase in moduli, and decrease in elongation properties as measured by mechanical property assessments.

Table 2: E'' and $\tan \delta$ values of the polybutadiene phase as a function of aging time at 90°C

Aging period (hours)	90°C		120°C	
	$\tan \delta$	Loss modulus (E'')	$\tan \delta$	Loss modulus (E'')
0	-86.02	-85.09	-86.02	-85.09
168			-78.74	-77.27
336	-85.27	-85.82	-75.64	-72.31
504			-80.91	-80.91
672	-85.22	-85.78	-80.87	-80.87

Initial assessment of ABS aged at 90°C and 120°C by positron annihilation spectroscopy (PALS) supports the results obtained by both physical and mechanical assessments. PALS enables non-destructive measurement of the free volume in a polymer, by discharging positrons into the material and monitoring their lifetimes (Ramani et al. 1995). PALS indicates that for ABS aged at 120°C, the number of free volume sites (I_3) and their mean radius (τ_3) decreases as a function of aging time (most likely in both phases), whereas for ABS aged at 90°C, free volume parameters do not significantly change with time.

4 Conclusions

The thermo-oxidative degradation of ABS at 120°C is a surface effect which forms a hard surface layer which allows initiation of cracks under impact and tensile loads. Thermal degradation of the bulk polymer in the absence of oxygen, deteriorates the bulk mechanical properties of the polymer, allowing propagation of the initial crazes from the surface of the polymer into the bulk, thus reducing the overall properties. Physical and mechanical assessment shows that polymer failure is greatly dependent on the thermo-oxidative degradation at the surface. Oxidation of the polybutadiene phase at the surface of ABS causes stress hardening, which in turn crazes the polymer surface. The critical depth of surface oxidation is measured to be approximately 0.06 mm. For greater depths of surface oxidation, impact or tension causes craze propagation through the bulk material, causing premature polymer failure. This effect is temperature dependent because, for exposures at 90°C and 100°C for the same periods of time, thermo-oxidative degradation in the form of mechanical property deterioration was significantly less. Further heat aging trials are being conducted to establish an Arrhenius temperature dependence for the thermo-oxidative degradation of ABS.

Degradation of ABS due to heat aging also occurs in the rigid SAN phase by physical aging. This physical aging is in the form of enthalpic relaxation which occurs due to heat aging of the SAN phase at sub- T_g temperatures. Physical aging causes an increase in the modulus of the SAN glassy phase, but in this case is not the controlling factor in the embrittlement of ABS.

5 Acknowledgments

The authors wish to thank Dr Anita Hill for the PALS analyses. The authors also wish to thank Euratech Limited for funding this work.

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83 THE EFFECTS OF ULTRAVIOLET RADIATION ON POLYCARBONATE GLAZING

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Abstract

Unstabilised bisphenol A polycarbonate samples were aged under UV radiation for periods of up to 3000 hours. The samples were then characterised using FTIR/ATR, GPC, SEM, microscopy, light transmission and colour changes. The changes in optical properties were related back to the changes in the polymer microstructure and composition.

In this study, the molecular weight of polycarbonate was seen to decrease during UV exposure, making it susceptible to embrittlement and surface erosion. This coupled with the discoloration of the polymer contributed to the loss of optical properties, including reduced light transmission, loss of gloss, increased haze, and discoloration. In the case of reduced light transmission and yellowing of polycarbonate, the degradation mechanisms were identified through infra-red spectroscopy. Photo-induced yellowing was accelerated by humidity, while the reduction in other optical properties was more severe in low humidity environments.

Keywords: Polycarbonate, photodegradation, photo-oxidation, molecular changes, physical characterisation, FTIR/ATR, surface analysis

1 Introduction

Polycarbonates are a very special class of engineering thermoplastic. They exhibit a unique combination of properties, for instance they have excellent toughness compared to other thermoplastics, have high transparency and dimensional stability over a wide temperature range, have good electrical properties and are lightweight. As a consequence, polycarbonates have established

their niche in a wide range of applications, including building and construction where they are often used as a lighter and tougher substitute for glass or metal, e.g. roofing and glazing.

However, bisphenol A polycarbonate is susceptible to photodegradation under ultraviolet (UV) radiation, which causes breakdown of the material, leading to yellowing, loss of toughness and embrittlement. In this paper, the authors look into the effects of UV on polycarbonate degradation, with the aim of understanding some of the relationships between chemical and physical changes taking place in polycarbonate and how the overall performance of the material is affected.

1.1 The effect of UV

When sunlight falls onto polycarbonate, the polycarbonate molecules absorb energy of certain wavelengths in the UV range, this is particularly so for the ester groups and the aromatic rings which are strong UV absorbers. The absorbed energy can cause the rupture of covalent bonds initiating photo-oxidation and photo-Fries reactions (Factor 1996).

Both mechanisms are distinct, i.e. they follow their own routes and have distinct intermediates and reaction products. The mechanisms are also wavelength dependent, i.e. one reaction predominates over the other depending on the wavelength of the incident radiation (Andrady et al. 1992). Photo-Fries is triggered by $\lambda < 310$ nm and photo-oxidation is favoured by wavelengths in the 290–350 nm range. In the environment, both mechanisms can take place and compete with each other (Rivaton 1995).

1.2 Photo-fries

The photo-Fries reaction is promoted by radiation of short wavelengths and does not require oxygen to occur. In the photo-Fries mechanism, the energy absorbed from UV radiation promotes the scission of the carbonate linkage, forming two primary free radicals (Rivaton 1995). The free radicals then rearrange to form phenyl salicylates and dihydroxybenzophenones, and other groups such as dihydroxybiphenyl and dihydroxydiphenyl ether groups. Oxidation of these groups results in substances such as ortho-dihydroxybenzophenone or diphenoquinone which absorb in the blue range, and are responsible for the yellowing observed in photodegraded samples (Andrady et al. 1992; Clark and Munro 1984; Pryde 1984). The reaction mechanism is shown in Figure 1.

The products formed during photo-Fries reactions can also be photo-oxidised, yielding secondary and tertiary products of lower molecular weight.

1.3 Photo-oxidation

For samples exposed to sunlight and air, photo-oxidation has been reported to be the predominant reaction, which occurs via a three-stage reaction (Clark and Munro 1982; 1984; Pryde 1984). In photo-oxidation (shown in Figure 2), the UV radiation is absorbed by impurities in the polymer, and as a result hydrogen atoms are abstracted from the methyl groups, forming a free radical and thus initiating a chain scission reaction which propagates the reaction. In the presence of oxygen, the methyl side chains are photo-oxidised into hydroperoxide intermediates, which

are transformed into tertiary alcohols and ketones. Further photo-oxidation of the products and of the aromatic rings also contributes to the yellowing of the polymer. Moreover, the free radical reaction is self-propagating until no more active sites are available or no oxygen is present to further the reaction.

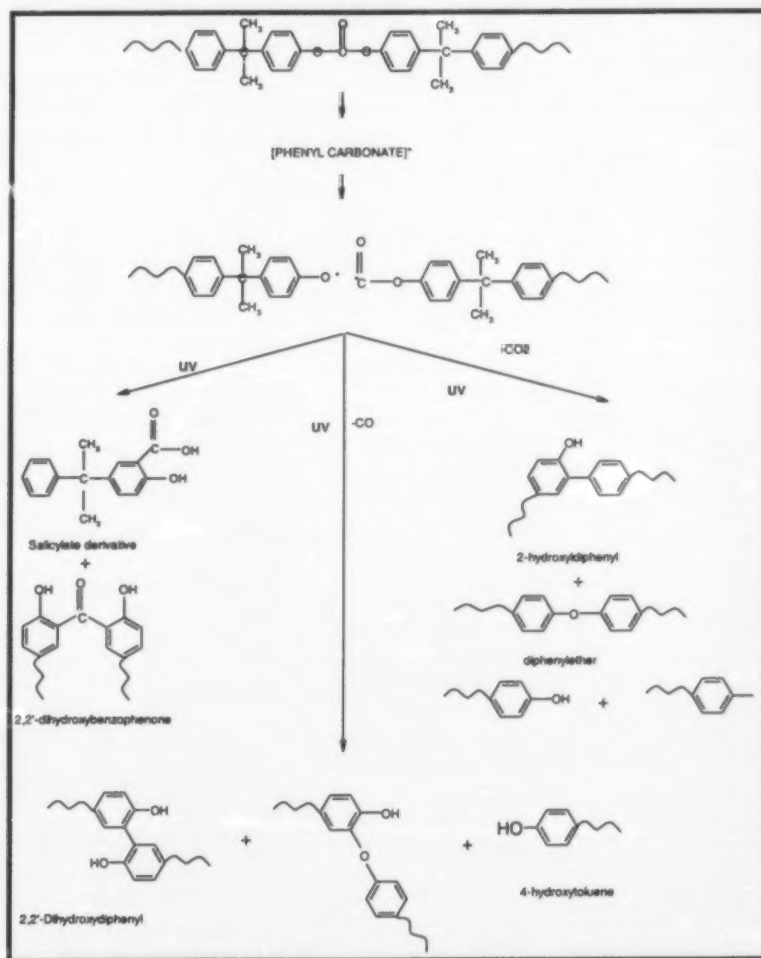


Fig. 1: Photo-fries reaction (source: Rivaton 1995)

The photo-oxidation of the aromatic rings has also been discussed to be a viable cause of yellowness, according to a study of model compounds, such as m-

BPO (3,3'-dihydroxy-diphenyl ether). When exposed to UV radiation under artificial and natural weathering conditions, these compounds showed yellowness index changes (Factor et al. 1987).

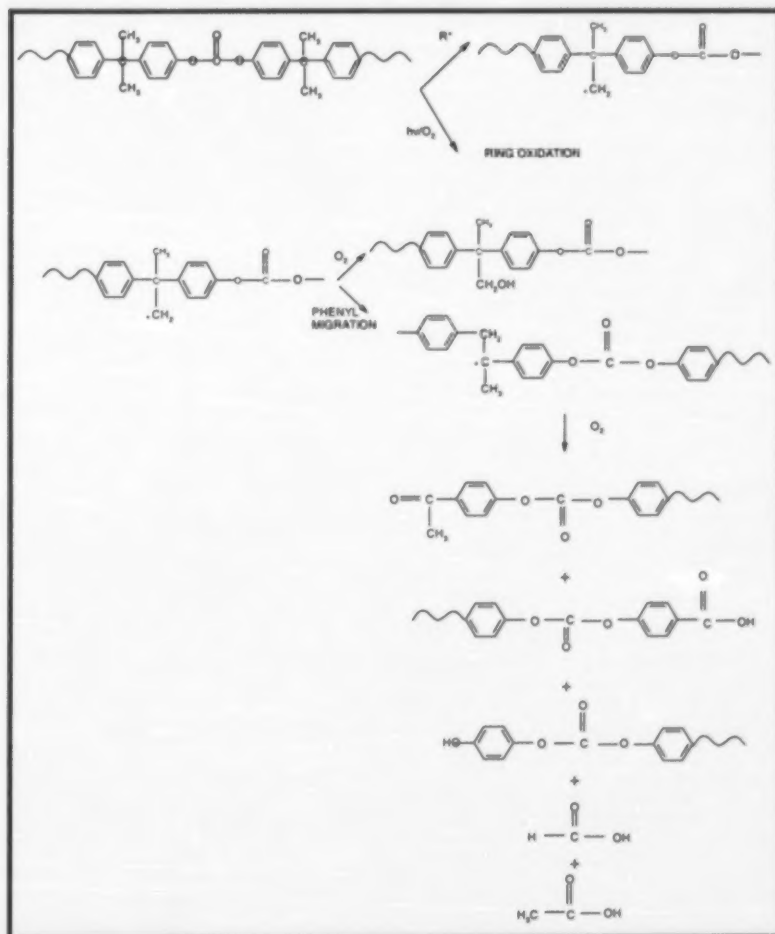


Fig. 2: Photo-oxidation of polycarbonate (source: Rivaton 1995)

In photo-oxidation, chain scission is favoured over cross-linking, particularly near polymer chain ends resulting in the formation of low molecular weight compounds. At short wavelengths, degradation free radicals tend to recombine into products, while under long wavelengths they react separately

forming structures with phenolic end-groups (Dilks 1989; Gorelov and Miller 1978).

Factor and Chu (1980) identified dual mechanisms acting in polycarbonate degradation. They noted that after irradiation of polymer films, two layers were formed – a superficial layer, brown and strongly oxidised, and beneath it a yellow layer. The superficial layer was attributed to photo-oxidation by short wavelengths, because the outer skin of the polymer would absorb the shorter wavelengths. Longer wavelengths would penetrate further into the polymer and promote the formation of the second layer. The degradation mechanism could thus be explained by a combination of photo-Fries followed by photo-oxidation at the surface layer caused by short wavelengths (<310 nm) and photo-oxidation caused by longer wavelengths in the inner layer. Hoyle et al. (1993) and Rivaton (1995) have verified that photo-Fries products could act as sites for photo-oxidation and be destroyed in the process, which would explain why only photo-oxidation and no photo-Fries products were identified in the polymer after long exposure periods.

1.4 Photobleaching

Degradation products that form from the exposure of polycarbonate to UV radiation can favour or hinder reactions that occur with further exposure, e.g. phenyl salicylate was reported to act as a photosensitiser, while dihydroxybenzophenone inhibits photo-oxidation (Rivaton 1995).

Rivaton (1995) reported that when yellow photolytic species were formed at very low concentration they could photo-induce the radical oxygenation of polycarbonate. On the other hand, as the concentration increased, these coloured species could also shield the polymer by decreasing the penetration of longer wavelength UV ($\lambda >310$ nm) into the inner polymer layers, thus reducing photo-oxidation. Further, photo-oxidation of the degradation products could also contribute to the destruction of coloured species and the formation of smaller polar molecules – a process known as photobleaching.

Andrady et al. (1992) reported the occurrence of photobleaching as the photodegraded polymer was exposed to longer wavelengths ($\lambda \sim 340$ nm). Photobleaching is in general observed after extensive exposure, when the rate of destruction of coloured species outgrows their rate of formation. However, it does not restore the damage caused by degradation.

1.5 Contribution of humidity

A review of the literature on the effect of humidity on polycarbonate discoloration has uncovered a number of conflicting observations. Polycarbonate is generally known to be inert to water at ambient conditions. It only undergoes hydrolysis under high pH (basic) conditions coupled with elevated temperatures. In such cases, the reverse of the condensation mechanism occurs and the polymer reverts to its original constituents. Under normal atmospheric conditions, the low reactivity of polycarbonate is attributed to the absence of polar groups in the molecule.

When photo-induced reactions under UV radiation occur, new degradation products are formed, and their reactivity to water could affect the polymer's

susceptibility to UV. Some researchers have observed that humidity accelerates the process of photo-yellowing of polycarbonate (Rivaton et al. 1986).

Conversely, Factor and Chu (1980) and Pryde (1985) observed that high humidity levels slowed the yellowing process. Pryde (1985) observed that for samples exposed to $\lambda > 284$ nm, relative humidity retarded the onset of degradation. This effect was more easily perceptible in samples irradiated in nitrogen where the controlling mechanism would have been photo-Fries. Pryde attributed the decreased reaction rate to the reaction between water and the excited species formed by photo-Fries. The hydrogen in water would be abstracted by the excited species and react with it, thus preventing the propagation of the reaction in the polymer molecule, decreasing the initial scission to radicals.

Rivaton et al. (1986) argued that chain scission could favour the hydrolysis of the polymer during exposure to water for long periods of time. Although the photo-oxidation (chain scission) products do not react directly with water, the chain breaking resulted in the formation of polar groups that favoured hydrolysis, releasing monomers and oligomers, which offer less resistance to oxidation compared to the original polycarbonate units (Rivaton et al. 1986). Thus the photo-oxidation rate is increased in the presence of high humidity levels. This would explain the observation by Factor and Chu (1980) that the exposed surface is more susceptible to hydrolysis than the bulk of the polymer.

In contrast to the effect of humidity on polymer degradation, several authors have discussed the solubility of the degraded species. Pryde (1984) suggested that the onset of photodegradation via a photo-Fries reaction (short wavelength) yields products which are hydrophobic and thus these products would not react directly with water. In contrast, for photo-oxidation at short wavelengths, as discussed by Clark and Munro (1983) polar products were formed that could be dissolved in water and leached out of the polymer. This would suggest that photo-Fries reactions have not occurred in this case, or that if they did occur, further photo-oxidation reactions occurred to convert the photo-Fries products to soluble products.

2 Experimental

Evaluation of the effect of UV radiation on polycarbonate sheets was achieved by aging the samples using an air-rich environment and a source of UV that simulated the sunlight spectra. Commercial Lexan® bisphenol A polycarbonate film, manufactured by GE Plastics, was used in this work. The samples were 1 mm thick and unstabilised (Mw ~60,350; Mn ~32,300).

The samples were exposed in a QUV Panel with UVA-340 nm fluorescent lamps. The emission spectra is very similar to that of sunlight for $\lambda < 340$ nm. The temperature was set at $40 \pm 2^\circ\text{C}$ and the relative humidity was either 0 or $42 \pm 5\%$. UV exposure was continuous for intervals up to 3000 hours.

Infra-red analysis was performed in attenuated total reflectance mode (ATR) using a Perkin Elmer 2000 FTIR, with microscope and a germanium tip crystal. The micro-ATR was adequate for measuring the external degraded layer without interference on the spectrum from the bulk polymer.

Surface analysis was performed using a scanning electron microscope (JEOL 35CF), source voltage 25 kV. The samples were coated in gold using sputtering conditions of 18 mA for 60 s. Microscopy was performed using a confocal laser microscope.

The molecular weight was determined by dissolving the samples in tetrahydrofuran AR grade to obtain a 1%w/v solution. The solution was prefiltered (pore size 0.5 μm) prior to analysis. The analysis was conducted in a Waters ALC 26 Gel Permeation Chromatograph (GPC) using narrow polystyrene standards and an injection rate of 1 mL/min.

Light transmission and haze were measured with a Gardner UX-10 Hazemeter, according to AS/NZ 4257.7 (Standards Association of Australia 1987) and ASTM D 1003 (American Society for Testing and Materials 1997a). Gloss was measured with a Gardner BYK Tri Gloss Meter at a 60° angle (American Society for Testing and Materials 1997b).

Discoloration was measured on specimens that were conditioned at $23 \pm 2^\circ\text{C}$ and $50 \pm 5\%$ relative humidity for a period of 48 h prior to each test. The yellowness index (YI) of the samples was measured with a Minolta Chroma Meter CR-200, according to ASTM D 1925-70 (American Society for Testing and Materials 1970).

3 Results and discussion

3.1 Effect of UV on molecular level

In both photo-Fries and photo-oxidation degradation there are three main groups in the polycarbonate molecule that could act as initiation sites, viz.:

- Carbonate groups (photo-Fries reaction).
- Gem-dimethyl group (photo-oxidation).
- Aromatic rings (ring oxidation).

As the degradation progresses, it is expected that photo-oxidation of the primary and secondary products will occur and thus photo-oxidation will predominate, accompanied by chain scission and the formation of low molecular weight compounds. Thus, it is possible to monitor such alterations through determination of functional groups and molecular weight.

Figure 3 shows the ATR spectra of polycarbonate specimens aged for different time periods under UV. In this figure, a broad peak at 3200 cm^{-1} appeared upon exposure to UV radiation. Such a peak indicates the presence of bonded hydroxyl (e.g. in carboxylic acids). As exposure proceeded, broad peaks at 2900 and 2600 cm^{-1} were formed, masking the absorption of the aromatic species. The carbonate peak (1767 cm^{-1}) decreased with exposure. At the same time, the absorption in the carbonyl region ($1760\text{--}1550\text{ cm}^{-1}$) increased with new peaks formed at 1701, 1685, 1596 and 1431 cm^{-1} . This is evidence for the formation of a range of photo-oxidation products, with substituted phenols, aromatic esters, aromatic aldehydes, heteroaromatic structures, carboxylic acids and aliphatic esters (1740 cm^{-1}) absorbing in this region.

The decrease in the carbonate peak at 1767 cm^{-1} is a sign that photo-Fries reactions took place, because carbonate sites act as initiation sites for these

reactions. The gem-dimethyl peaks at 1390, 1380 and 1360 cm^{-1} were overtaken by the broadening of the carbonyl peaks (1500–1780 cm^{-1}), although there does not seem to be a significant reduction in the concentration of the gem-dimethyl groups. Thus, it is unlikely that the dimethyl groups were the main initiation sites for the reaction. However, we cannot exclude the possibility that a few gem-dimethyl groups might also have served as initiation sites during photo-oxidation reactions. Ring oxidation of aromatic rings is also a dominant reaction because the absorption in the 900–800 cm^{-1} (aromatic C-H) region decreased, indicating an increase in substituted phenolic species.

In summary, the spectra obtained support the affirmations of Rivaton (1995) that UV degradation was initiated by photo-Fries reactions and that as the degradation progresses these reactions are superseded by photo-oxidation reactions. Additional evidence of this is provided by the GPC measurements discussed in the following paragraphs. However, further confirmation of the specific reaction route details is still under investigation.

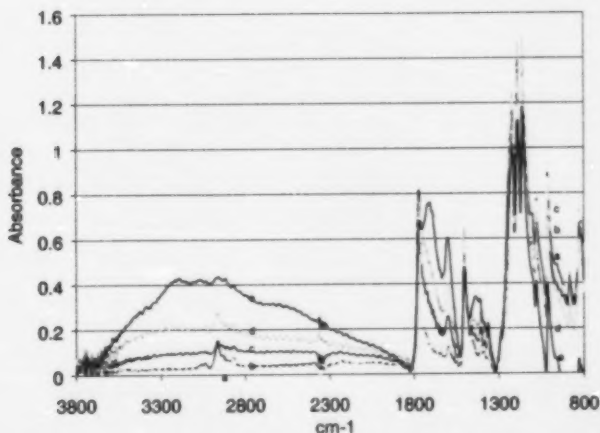


Fig. 3: ATR spectra of polycarbonate exposed to UV at 40°C, 42% relative humidity: (a) control (0 h); (b) 528 h; (c) 1320 h; (d) 1680 h and (e) 2352 h

The molecular weight measurements of polycarbonate exposed to UV radiation indicate the predominance of chain scission rather than cross-linking during the consequent photo-oxidation. Cross-linking is a characteristic of photo-Fries and a large polydispersity (M_w/M_n) ratio denotes its predominance. Thus, if cross-linking predominates there is an increase in the M_w/M_n value. In Figure 4 the average molecular weight and the number average of the samples decreased upon UV exposure, indicating a shift of the molecular weight distribution into lower molecular weight values. For instance, after 2000 h of UV exposure at 0% relative humidity, M_w decreased by 8.9% and M_n decreased by 10.9%, and after 3050 h at 42% relative humidity, M_w was reduced by 3.2%, while M_n was

reduced by 6.0%, indicating the occurrence of chain scission in the polymer. The changes were more severe for the 0% relative humidity sample, as discussed later. Overall, the polydispersity values remained around 2, while in the case of cross-linking, polydispersity would have been expected to increase significantly.

The changes detected in this work were not as dramatic as those of other authors. For example, Sherman et al. (1982) measured a 30% reduction in M_w over 2000 h, but this can be accounted for by the fact that their samples were 0.2–0.3 mm thick, while the samples in our study were 1.0 mm thick. Thus, the overall degradation of their samples would have been more uniform. Chain scission and the formation of smaller molecules would lead to the embrittlement of the polymer. Thus, the surface would be prone to erosion, possible craze initiation, embrittlement and crack propagation. After exposure, some samples were cut with shears, and it was observed that as the time of UV aging increased, the ductility of the polymer decreased and a broader network of cracks extended into the polymer.

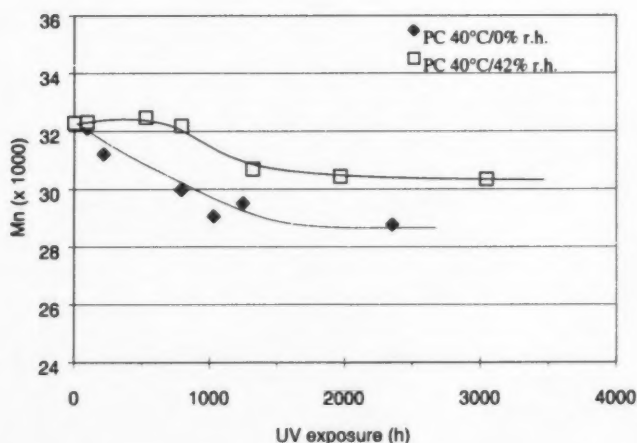


Fig. 4: The effect of UV radiation on molecular weight number and average

3.2 Effect on polymer physical properties

The optical properties were evaluated because of the widespread application of polycarbonate as a glazing material. In such applications uniform light transmission and clarity are fundamental. The light transmission through a material is influenced by factors such as the colour of the sample, the presence of light-scattering bodies in the sample, and the sample surface.

For polycarbonate, the colour of the samples changed upon exposure to UV, initially from a very faint yellow to a stronger yellow, as observed by measuring the change in the yellowness index of the material, as shown in Figure 5. As exposure continued, photo-bleaching occurred, as indicated by the reduction in the value of the yellowness index.

The colour changed progressively and showed a self-accelerating stage, which was characterised by a higher gradient. The curve could be divided into three stages. For samples exposed at relative humidities of 42% and 0% respectively, the first stage was up to 500 h (42%) and 800 h (0%), the second self-accelerating stage was at 500–800 h (42%) and 800–1400 h (0%), and the third started from 800 h (42%) and 1400 h (0%). Examination of the results for the sample exposed at 42% indicates that it underwent photobleaching after 1500h.

During the first stage, the degradation reactions initiated and Photo-Fries reactions would predominate. In the second stage, the rate of yellowing increased markedly, and in the third stage, the rate of yellowing decreased and a maxim YI value was reached. The changes in the rate of yellowing can be explained by competing degradation reactions. As initial degradation proceeded, the increase in carbonyl groups favoured the absorption of UV. This accelerated the photoreactions and caused the change in slope at stage 2. The photo-oxidation reaction became the predominant reaction after initiation by photo-Fries, then the acceleration of yellowing rate observed in this stage for 42% relative humidity could be explained by Clark and Munro (1983), i.e. that humidity may have assisted in the removal of degradation products from of the polymer, favouring further oxidation.

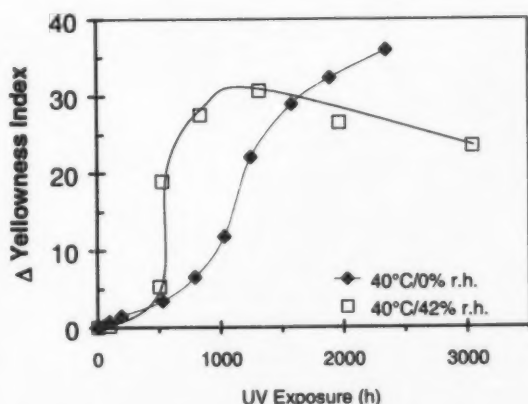


Fig. 5: Effect of UV on the yellowness index of polycarbonate

In the third stage, photobleaching would start to occur, which would mean that the coloured species were being destroyed, i.e. the rate of formation of yellow species was lower than the rate of destruction of the same species. If we assume that the maximum concentration of yellow species is reached, further oxidation of the photoproducts would destroy the conjugated sequences, causing bleaching.

By comparing the results obtained from analysis at low and medium humidity, it was seen that the onset of yellowing was favoured by the increase in relative humidity. Thus, the phenomena reported by Factor and Chu (1980) and

Pryde (1985) that humidity prevents degradation by reaction with free radicals was not observed in this work. It is possible that at higher relative humidity levels (>80%) such an effect might be valid, however, this could need to be confirmed. Our observations do corroborate the results from Rivaton et al. (1986) that humidity accelerates the yellowing process.

At stage 3, a maximum YI was reached for a relative humidity of 42%, whilst for the samples at a relative humidity of 0% the YI kept increasing. Based on the discussion in section 1.2, a possible explanation for this is that the acceleration of photo-oxidation on the outer layers by the higher levels of humidity increased the formation of UV-absorbing species at the polymer surface. This layer then screened the lower wavelength of UV radiation and prevented it from penetrating into the bulk of the polymer and promoting further photoreactions (Factor and Chu 1980). This concept was also discussed by Rivaton (1995), who suggested that as the saturation concentration was reached, the surface layer became fully oxidised. The chromophoric products would then prevent the propagation of the UV radiation into the inner layers of the polymer, thus acting as a barrier against part of the UV range. This would prevent photoreactions in the bulk of the polymer.

For the specimen at a low humidity level, yellowing would occur at a slower rate, allowing the longer wavelengths to penetrate further into the polymer. This would allow a thicker layer of chromophores to be formed, which would eventually allow more chromophores to form than for exposure at high humidities.

As shown in Figure 6, the light transmission of polycarbonate decreased linearly with increased exposure to UV radiation. The reduction in light transmission observed after 2000 h was approximately 4% and 5% for samples aged at 42% and 0% relative humidity, respectively. As the sample turns from transparent to yellow, the polymer absorbs more visible light in the blue range, reducing the total light transmitted through the sample. Therefore, yellowing is partly responsible for the reduction in light transmission. However, other factors such as haze and gloss have to be considered.

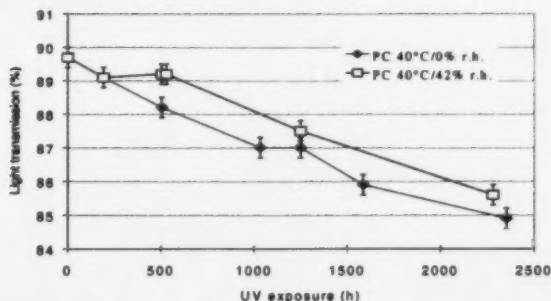


Fig. 6: Effect of UV on the light transmission of polycarbonate

Figure 7 shows that an increase in haze occurs in the samples. The haze increased by slightly less than 6% over a 2000 h period. Haze is a measure of the light scattering of the sample, therefore if the surface becomes uneven or if light-scattering agents are formed in the specimen, the haze values increase and the

light transmission values decrease. The samples aged at 0% humidity had a higher haze value than samples aged at 42%, after 1000 h, which would indicate that the phenomenon responsible for haze was more severe at lower humidity.

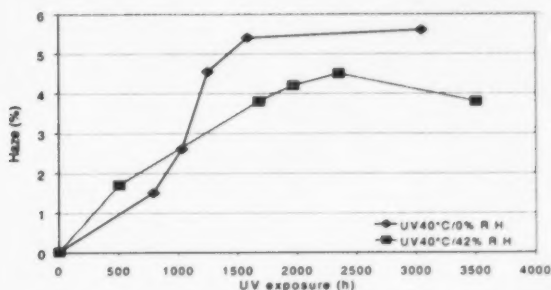


Fig. 7: Effect of UV on the haze of polycarbonate

As shown in Figure 8, the level of surface gloss of the specimens decreases with exposure to UV, with the reduction being approximately 5% after 2000 h exposure. This would cause more light to be scattered diffusively at the surface of the sample, thus reducing light transmission and increasing haze. The loss in gloss can be explained by examination of SEM micrographs as shown in Figures 9 and 10. It can be seen that the surface of the exposed sample has an eroded appearance, whilst the control sample exhibits a more uniform and smooth surface.

The observations for light transmission, haze and gloss can be related back to our earlier discussion, where at 0% relative humidity the degradation was more severe for all properties measured except yellowing. As discussed above, the reduction in Mw and Mn was larger for samples at 0% relative humidity than for those at 42% relative humidity, indicating that more chain scission and polymer degradation had occurred. As also discussed, the depth of penetration of UV radiation at 0% would be deeper than at 42%, the oxidised layer formed would be thicker and the number of chain scissions per polymer cross-section would be larger. The thicker layer of degraded polymer formed at 0% relative humidity would interfere more strongly with the path of light than the thinner layer formed at 42% relative humidity, giving lower levels of light transmission and increased levels of haze.

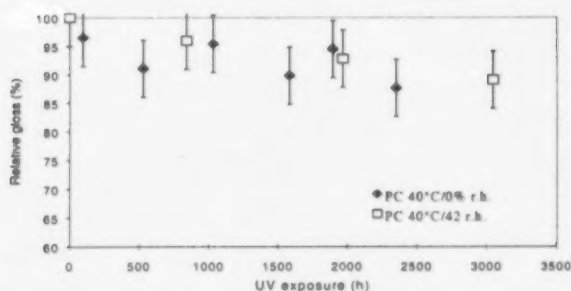


Fig. 8: Effect of UV on the gloss of polycarbonate (relative to the gloss of the control sample)

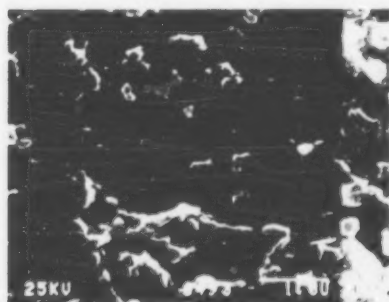


Fig. 9: SEM of polycarbonate exposed to UV for 2350 h

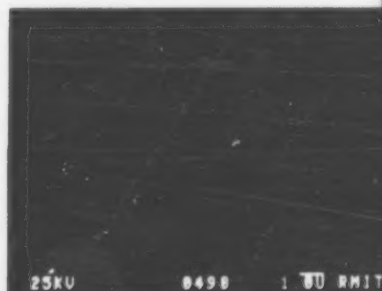


Fig. 10: SEM of polycarbonate control

4 Conclusions

The effect of a range of atmospheric degradation agents on the performance of polycarbonate was investigated in this study. The aim was to determine the effect of these agents, investigate the relationship between physical and chemical changes, and to investigate their degradation mechanisms. The conclusions from our study are:

- Under UVA-340 nm lamps, the mechanism of degradation was identified to be a combination of photo-Fries and photo-oxidation, the later being the predominant reaction over time. This resulted in discoloration, chain scission and consequently a reduction in the molecular weight of the polymer.
- Chain scission at the polymer surface caused embrittlement of the surface, making it more susceptible to erosion. As a result, the polymer surface became uneven, which increased the scattering of light on the polymer surface, reducing gloss and contributing to the increase in haze of the polymer. The

increase in haze and the absorption of visible light due to yellowing decreased the total amount of light transmitted through the polymer.

- Humidity affected the competition between photo-Fries and photo-oxidation, which led to differences in the depth of the degraded layer on polycarbonate. The effect was particularly evident through the changes in yellowing. Yellowing showed that increased humidity affected the rates and mechanisms of the photoreactions.
- At high humidity, yellowing was initially accelerated (the initial YI values obtained were higher than at the low humidity). It then reached a maximum value with time and was followed by photobleaching.
- At low humidity, deterioration in gloss, light transmission, haze, discoloration and molecular weight were more severe. Yellowing increased over time, finally reaching higher values than for the samples at higher humidity.

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